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Use of NDE to Evaluate Reflection Cracking in Airfield Pavements

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This study reports on the possible use of nondestructive testing equipment to evaluate the potential of reflection cracking in airfield pavements. It also summarizes the state of the art of reflection cracking analysis and methods for controlling reflection cracking. The proposed method of analysis consists of testing portland cement concrete slabs in a curled-up condition at slab centers and slab corners. Increased tensile strains at the bottom of the overlays at slab corners are related to the number of traffic repetitions to predict reflection cracking life. Although the method appears promising, further study is recommended to further extend the limited data base presented here and gain further insight into the nature of mechanisms which cause reflection cracking. Keywords:

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PREFACE

This report was prepared by the University of Washington, Seattle, Washington 98195, under contract F08635-84-K-0145, for the Air Force Engineering and Services Center, Engineering and Services Laboratory, Tyndall Air Force Base, Florida.

This report summarizes work done between February 1984 and August 1984. Guidance in the selection of test sites and field testing was provided by the Washington State Department of Transportation (WSDOT). Mr. James G. Murfee was the AFESC/RDCP project officer.

This report provides a state-of-the-art review of the mechanisms associated with and methods of controlling reflection cracking in asphalt concrete overlays of portland cement concrete. It also provides a proposed method of evaluating the potential of reflection cracking using nondestructive testing data. The report does not constitute an endorsement or rejection for any piece of equipment or product for Air Force use nor can it be used for advertising any product.

Special acknowledgment is due to Mr. Newton C. Jackson of the WSDOT Materials Laboratory for his participation in the conduct of this study. Ms. Beverly Odegaard prepared the manuscript and contributed to the editing.

This report has been reviewed by the Public Affairs Office (PA) and is releasable to the National Technical Information Service (NTIS). At NTIS, it will be available to the general public, including foreign nationals.

This technical report has been reviewed and is approved for publication.

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SECTION I

INTRODUCTION

BACKGROUND

Reflection cracking in pavement overlays was identified as a major problem as early as 1932 at the annual meeting of the Transportation Research Board (Reference 1). The problem remains, along with a continued interest in the use of pavement overlays for the rehabilitation of existing pavements. Many techniques have been tried to prevent or reduce reflection cracking with varying degrees of success. The inconsistency in the success of these techniques may be partly due to a lack of analytical methods for evaluating the problem (or reluctance to implement existing analytical approaches).

Numerous methods of evaluating the potential for reflection cracking have been proposed. These include elastic analysis (References 1 through 4), elastic fracture mechanics (References 5, 6, 7) and viscoelastic fracture mechanics (References 8, 9, 10). The mathematics required for these solutions range from simple algebra to complex finite element analysis and all have theoretical validity. However, none of them have been widely accepted for routine use by pavement engineers.

Thus, the general problem of reflection cracking in asphalt overlays seems to lie in the development of usable procedures for evaluating the potential of reflection cracking. The following factors should be incorporated in such a procedure:

1. Material properties of the existing pavement,
2. Material properties of the overlay,
3. Material properties of interlayers or other systems proposed for reflection cracking control,
4. Structural integrity of the existing pavement,
5. Load-carrying capacity of the overlayed pavement,
6. Design life of the overlayed pavement, and
7. Environmental factors for the area under consideration.

The method used in this study addresses many of these factors for asphalt concrete overlays on portland cement concrete (PCC) pavements.

Basically, the method used data from a nondestructive deflection measuring device (Falling Weight Deflectometer) and layered elastic computer programs to evaluate the material properties and structural integrity of the existing pavement. Total strain in the asphalt concrete overlay was calculated by summing the strain due to curling and traffic load on the PCC slab. It may be possible to ascertain the amount and severity of reflection cracking from the value of total strain in the asphalt overlay.

The U.S. Air Force has sponsored previous research in reflection cracking (References 11, 12, 13). These studies focused primarily on methods to prevent or reduce reflection cracking. The current study differed from the previous ones in that it was an attempt to analyze the mechanisms of reflection cracking. By understanding these mechanisms, it may be possible to judge the benefits from the various crack reduction systems.

OBJECTIVES

The cbjectives of this study were to:

1. Conduct a review and summarize available information on reflection cracking mechanisms and treatments.
2. Use the Falling Weight Deflectometer (FWD) and layered elastic theory computer programs to evaluate a pavement overlay design concept which would minimize reflection cracking of asphalt concrete overlays on portland cement concrete pavement.

SCOPE

The scope of this study may be divided into two major components: (1) state-of-the-art review and (2) data collection and analysis for the proposed method. The state-of-the-art review consisted of reviewing available methods for analyzing and reducing reflection cracking in asphalt overlays.

The second part of the study involved:

1. Obtaining deflection and traffic data from overlayed PCC pavements,
2. Analyzing the data using layered elastic computer programs, and
3. Correlating the data with limiting strain criteria for the asphalt overlays.

REPORT ORGANIZATION

The remainder of this report is divided into the following sections:

II. Existing Methods of Analysis

III. Methods of Controlling Reflection Cracking

IV. Proposed Method of Analysis

V. Results and Discussion

VI. Conclusions and Recommendations

The Sections II and III present a state-of-the-art review. Section IV gives a detailed description of the proposed method of analysis using nondestructive test (NDT) data with layered elastic theory. Section V summarizes the findings of this study with respect to the proposed method of analysis. Conclusions and Recommendations in Section VI are based on the findings.

SECTION II

EXISTING METHODS OF ANALYSIS

INTRODUCTION

This section briefly describes the mechanisms of reflection cracking, as they are generally understood. It also presents methods to analyze pavement systems for potential reflection cracking. As stated earlier, the mathematics used in these methods range from simple algebra to finite element analysis. While most readers are familiar with fundamental algebra and calculus, a brief review of finite element analysis is presented for those not familiar with these. Since some analytical methods are based upon fracture mechanics, a brief overview of this area will give the reader a basic understanding.

FINITE ELEMENT ANALYSIS

The finite element method, simply stated, is a numerical procedure in which solutions are obtained for individual pieces of a particular domain. Solutions of each of these pieces are considered collectively to obtain an approximation of the system response. The method is a powerful tool in the analysis of situations involving complex geometry and loading. Cook (Reference 14) has written a text on the engineering applications of finite element analysis. This discussion is based on Cook's text.

Cook traces the beginning of finite element analysis to 1906 when Wieghardt (Reference 15) suggested the use of "lattice analogy" for the solution of continuum problems. In 1943, Courant (Reference 16) established many of the ideas used in finite element analysis. Argyris and Kelsey (Reference 17) and Turner, et al. (Reference 18) are generally credited as having made the greatest contributions to the finite element method in the mid-1950s.

To begin finite element analysis, a structure such as the pavement system shown in Figure 1.a. is subdivided into a mesh as shown in Figure 1.b. The rectangular areas are called elements and these are connected to one

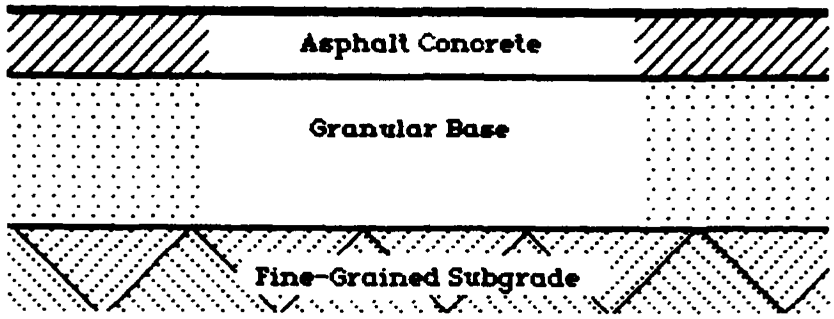

Figure 1.a. Schematic of Structural Section.

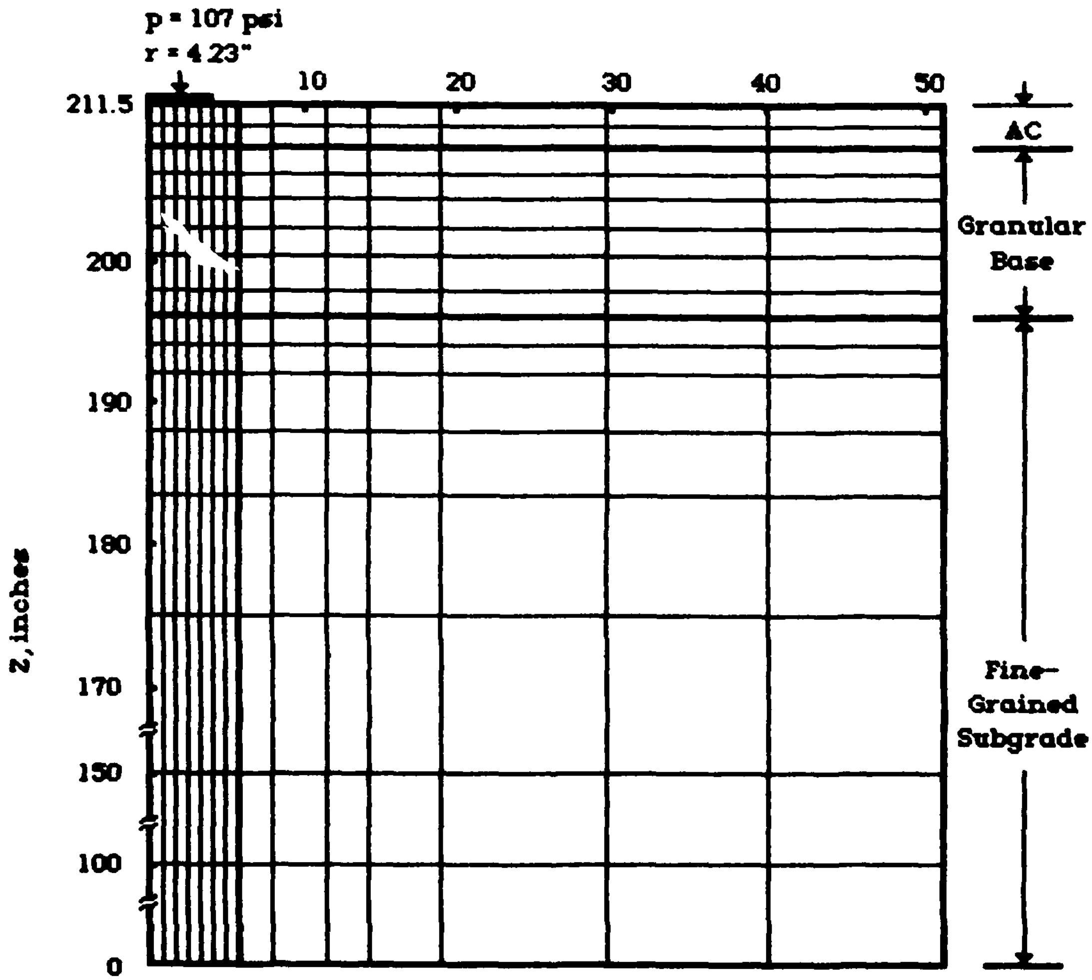

Figure 1.b. Finite Element Mesh for Structural Section

Figure 1. A Pavement Cross Section as Represented by Finite Element Mesh.

another by nodes (intersections of element boundaries). The stiffness at each nodal point is calculated by means of assuming displacement variation within the element, along with a knowledge of the stress-strain behavior of the element material. Equilibrium at each nodal point may be expressed by two equations which use displacements and stiffnesses to define nodal forces. The equations are used to solve for unknown displacements. Once the displacements at all of the nodal points have been calculated, the stresses and strains for each element may be computed.

The preceding discussion describes a two-dimensional analysis and a three-dimensional analysis may be described in a similiar fashion.

The concept of finite element analysis has recently been applied to pavement analysis. Computer programs for predicting pavement response to loads have been developed for asphalt concrete (References 19,20) and portland cement concrete (Reference 21). Finite element computer codes developed for reflection cracking analysis are discussed in References 3 through 7 and 9.

FRACTURE MECHANICS

Fracture mechanics provides a means for determining the causes of failure in materials which have been subjected to stresses lower than their design stresses. These failures begin with the presence of existing microscopic cracks in the material. As loads are applied to the structure, the cracks become larger,propagating through the structure until fracture occurs.

Griffith (Reference 22) is generally acknowledged for the original development of fracture mechanics in 1920. He assumed that fracture occurred in brittle materials when the rate of decrease in elastic strain energy due to an increase in crack length is equal to or greater than the rate of increase in surface energy at the crack tip. This implies that a rapid crack growth will occur when more elastic energy is released than can be stored on the crack surfaces. Griffith's theory describes the catastrophic failure commonly found in brittle materials.

Irwin (Reference 23) developed the stress analysis for crack growth as shown in Figure 2. Mode I is called the Opening Mode in which crack growth

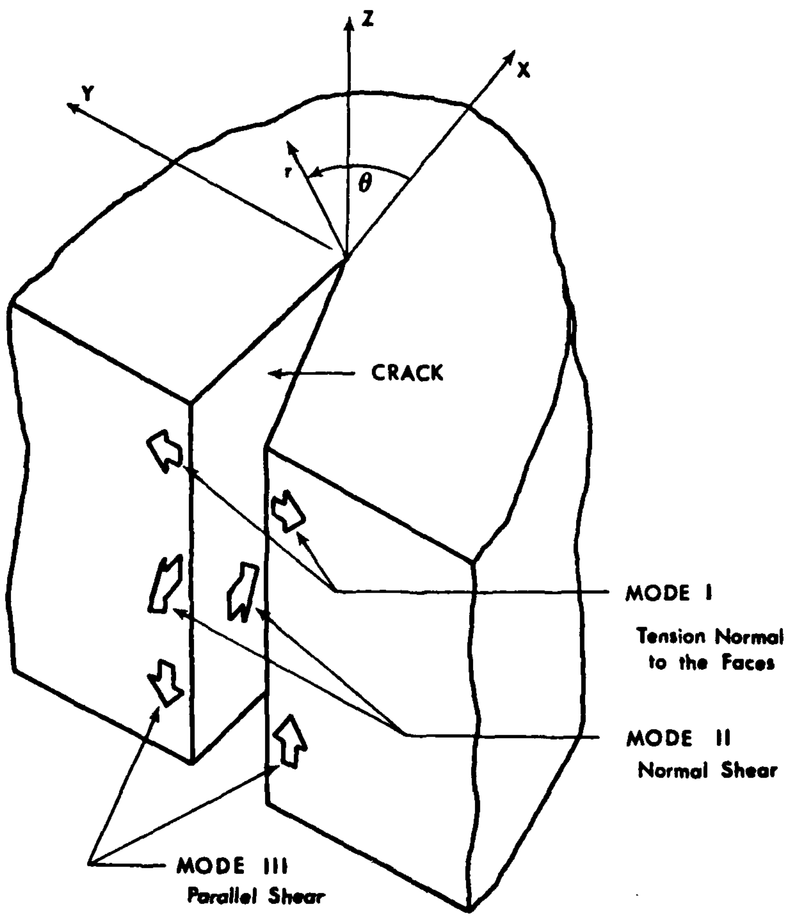

Figure 2. Modes of Crack Growth (Reference 7).

occurs due to tensile stresses. Mode II crack growth is due to normal shear stresses. Parallel shear stresses are responsible for Mode III. Figure 3 shows how the stresses appear at the leading edge of the crack. Modes I and II are normally of greater concern in pavement cracking than Mode III.

Irwin (Reference 23) used elastic theory to define the stresses and displacements in the vicinity of a crack tip. He employed a method developed by Westergaard (Reference 24) to represent the Airy's stress function in terms of complex variables which account for the crack boundary conditions. Mode I stresses at the crack tips are defined by the equations:

$$\begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \tau_{xy} \end{Bmatrix} = K_I (2\pi r)^{-1/2} \cos\frac{\theta}{2} \begin{Bmatrix} 1-\sin\frac{\theta}{2} \sin\frac{3\theta}{2} \\ 1+\sin\frac{\theta}{2} \sin\frac{3\theta}{2} \\ \sin\frac{\theta}{2} \cos\frac{3\theta}{2} \end{Bmatrix} \tag{1}$$

For Mode II, the following equations apply:

$$\begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \tau_{xy} \end{Bmatrix} = K_I (2\pi r)^{-1/2} \cos\frac{\theta}{2} \begin{Bmatrix} 1-\sin\frac{\theta}{2} \left(2+\cos\frac{\theta}{2} \cos\frac{3\theta}{2}\right) \\ \sin\frac{\theta}{2} \cos\frac{\theta}{2} \cos\frac{3\theta}{2} \\ \cos\frac{\theta}{2} \left(1-\sin\frac{\theta}{2} \sin\frac{3\theta}{2}\right) \end{Bmatrix} \tag{2}$$

For equations (1) and (2):

σ_{xx}, σ_{yy} = normal stresses in the noted directions,

τ_{xy} = shear stress in the noted direction,

r = radial distance from crack tips,

θ = angle of r from the direction of crack growth, and

K = stress intensity factor for crack growth mode.

The stress intensity factor, K, is used to define the stress field near the crack tip in consideration of load, geometry, and boundary conditions and is proportional to the force causing the crack growth.

Paris and Erdogan (Reference 25) developed procedures for evaluating fracture mechanics in a situation involving stable crack growth. In describing the rate of crack propagation, $\frac{dc}{dN}$, they found that in many materials the rate was proportional to K . Thus, Paris law states that:

$$\frac{dc}{dN} = A(\Delta K)^n \tag{3}$$

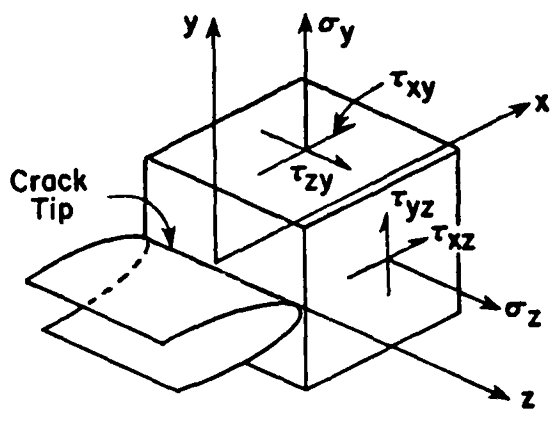

Figure 3. Stresses at the Leading Edge of a Crack (Reference 10).

Where: c = crack length,

N = number of load repetitions,

$\frac{dc}{dN}$ = increase in crack length per loading cycle,

K = stress intensity factor,

A = regression constant, and

n = regression constant (4 for Mode I fracture).

The application of this law depends on two important assumptions:

1. The material is homogeneous, isotropic, and elastic-plastic and
2. The size of the plastic zone at the crack tip is small relative to the crack size and overall dimensions of the system.

The equation for crack growth in visoelastic media as developed by Schapery (Reference 26, 27, 28) is similar in form to Equation (3).

MECHANISMS OF REFLECTION CRACKING

Bone and his co-workers (References 30, 31) acknowledged in the 1950s that the primary causes of reflection cracking were horizontal and differential vertical movements at existing cracks. Horizontal movements are normally associated with environmental factors, while vertical movements are attributed to both traffic loads and environmental factors.

Shrinkage of Asphalt Overlays

Environmental factors have two different effects on overlayed PCC pavements. One effect is the volumetric change which occurs in the overlay material. The expansion and contraction of asphalt concrete pavements produces the familiar problem of transverse cracking. Transverse cracking has been the subject of numerous studies and suggested design criteria (References 32 through 42). This problem is compounded in asphalt overlays of PCC due to the warping or curling of the underlying concrete slab.

Curling of PCC Slabs

Curling of a concrete slab occurs because of a temperature differential between the top and bottom of the slab. When the top of the slab is cooler than the bottom, it contracts more. This causes the corners to curl upwards producing a weakened condition at these points. The weight of the slab holds it in place and the curling is usually expressed in terms of stresses induced at the corners. Westergaard (Reference 43) first proposed a method of analy-

sis for curling in 1926. Bradbury (Reference 44) built upon Westergaard's work to develop practical solutions. Darter (Reference 45) recently developed another method which consists of regression equations developed from a finite element analysis.

Curling stresses contribute to reflection cracking through a combination of horizontal and vertical differential movements at joints. Horizontal movement of adjoining slabs causes stresses at the bottom of the asphalt overlay. As the temperature cools, the adjoining slabs contract, causing the joint to become wider and inducing a tensile stress at the bottom of the overlay. Vertical differential movement in the pavement due to curling stresses occurs due to the loss of support and load transfer at slab edges. As a traffic load moves from the interior of the slab to the edge, deflection becomes greater. A combination of increased tensile and shear stresses may be induced in the overlay.

Loss of Slab Support

Past pumping of a soil beneath a concrete slab will also contribute to reflection cracking in an overlay. Yoder and Witczak (Reference 46) have defined pumping as "the ejection of water and subgrade (or base) material through joints and cracks or at the pavement edge, caused by the deflection of the slab after free water has accumulated under the slab." As more supporting material is removed from underneath the slab, a void is formed at the crack, edge, or joint. This void is a zone of weakness. If remedial measures such as undersealing are not performed prior to overlaying the concrete pavement, differential vertical movement of the slabs will persist. Again, this will result in the development of increased tensile and shear stresses in the overlay.

DISCUSSION

In general, reflection cracking due to changes in volume of materials has been noted (References 47, 48, 49) to start with the development of small hairline cracks during the first cycle of cool weather after construction. Within a relatively short span of time (much less than the design life) these cracks may deteriorate to an unacceptable level. Mazjidzadeh and Suckarieh

(Reference 50) attached great importance to this mode of failure in their study.

McGhee (Reference 51) has demonstrated that the primary cause of reflection cracking may be vertical differential movement in PCC slabs in certain instances. This was demonstrated by the data presented in Table 1. These data were obtained using a Benkelman Beam with an 18,000-pound single-axle load applied to either side of joints in PCC overlaid with asphalt concrete. Differential vertical movements in slabs result in shear stresses developing in asphalt concrete overlays, and the magnitude of these stresses is related to the magnitiude of differential movement (References 52, 53).

METHODS OF ANALYSIS

The analytical methods presented here were developed by researchers at four different organizations. These were the University of California at Berkeley, Ohio State University, Texas A & M University, and Austin Research Engineers. More than one method each were developed at Texas A & M and Berkeley.

University of California at Berkeley

General Layered-Elastic Analysis

McCullough (Reference 2) chose layered theory over plate theory as a mathematical model for an overlay design system for the following reasons:

1. The ability to predict the state of stress in the surface layer.
2. The ability to realistically treat vertical stress.
3. The ability to use measurable material properties as an integral part of the analysis.

Furthermore, he chose to use general layered theory over the finite element method for the following reasons:

1. Finite element methods showed no improvement in results over conventional layered methods.
2. The use of extensive computer time with finite element methods.

McCullough began his analysis with Haas' (Reference 34) formula for approximating thermal stresses in asphalt concrete:

$$\sigma_{AC}(T) = \alpha_A \sum_{T_o}^{T_f} S(\Delta T)\ [\Delta T] \qquad (4)$$

TABLE 1. THE RELATION BETWEEN DIFFERENTIAL BENKELMAN BEAM DEFLECTIONS AND REFLECTION CRACKING (REFERENCE 51).

Differential Deflection* (in.)	Percent Joints Cracked, %			
	Route 460 Project		Route 13 Project	
	Fabric	Control	Sanded	Control
0	0	44	24	100
0.002	29	54	57	100
0.004	88	74	77	100
0.006	88	100	93	100
0.008	100	100	--	---

*Measurements were taken with the Benkleman Beam using an 18-kip (8172kg) single axle load.

Where: α_A = mean coefficient of thermal contraction of asphalt concrete *over the temperature range* ΔT *and*

S = mean stiffness of the asphalt concrete over the temperature range ΔT.

He used the following equation to describe the thermal forces transmitted from the concrete to the asphalt concrete:

$$P_{\Delta x} = \int_0^{x/2} [\tau_{\Delta x} (\bar{x}, T)] \cdot [W] d\bar{x} \tag{5}$$

Where: $\tau_{\Delta x}$ = shear stress at the asphalt concrete interfaces, a function of joint spacing and temperature,

$\bar{x}$ = joint or crack spacing, and

W = pavement width.

McCullough used a similar equation to describe forces transmitted to the subgrade or base from the PCC.

Using work done by Haas (Reference 34), Finn (Reference 54), and Jones and Hirsch (Reference 55), McCullough developed the following thickness criteria for asphalt overlays of PCC:

$$D_A \geq \frac{[R_2 \frac{0.482}{F^{2.03}} \cdot \alpha_c \cdot E_c \cdot D_c] \Delta T_c}{f_{AC}(T) - \alpha_A \sum_{T_o}^{T_f} s(\Delta T)[\Delta T_A]} \tag{6}$$

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Where: D_A = thickness of asphalt concrete layer,

R_2 = constant between 0 and 1.0,

F = resistance factor for the supporting material,

α_c, α_A = coefficient of thermal contraction for PCC and asphalt concrete, respectively,

E_c = concrete modulus of elasticity,

D_c = thickness of PCC,

ΔT_c, ΔT_A = temperature differential experienced by the PCC and asphalt concrete, respectively,

$f_{AC}(T)$ = strength of the asphalt concrete as a function of temperature, and

$S(\Delta T)$ = mean stiffness of the AC over the temperature range ΔT.

This criterion ensures that volume change fracture will not occur in the asphalt overlay. It does not address the problem of shear fracture.

Elastic Finite Element Analysis

Coetzee (Reference 3) and Monismith and Coetzee (Reference 4) described a procedure which considers both horizontal and differential vertical movements. Coetzee (Reference 3) outlined the following criteria for developing a method to evaluate the potential of reflection cracking:

1. It should be simple to use and understand,
2. It should be able to consider alternatives to retard or eliminate reflection cracking, and
3. It should be consistent with theoretical predictions of reflection cracking.

To simulate an overlayed PCC pavement, Coetzee used a testing system consisting of two concrete slabs with a specified joint opening. These slabs were placed on springs (elastic foundation) and overlaid with asphalt concrete or an asphalt-rubber membrane and asphalt concrete. Cyclic loads were applied over the midpoint of the joint. Cracks at the side of the asphalt concrete were measured as they propagated. This system was found not to accurately model an in-service pavement.

Monismith and Coetzee (Reference 4) proceeded to develop a theoretical method of analysis. This procedure uses finite element analysis to define the pavement section. A stress analysis is performed to ascertain the response of the overlay under a particular load. The stress and strain values in the interlayers are evaluated to see if they exceed predetermined limiting criteria developed for the interlayer material. Next, the overlay is evaluated for possible premature cracking, i.e., fatigue cracking. Overlay parameters and thickness may affect the response of the interlayer. Thus the interlayer must be included in the overlay thickness evaluation. Figure 4 shows a flow chart of the design subsystem.

Coetzee (Reference 3) found that one of the most important parameters governing reflection cracking of asphalt overlays of PCC with no interlayers was the ratio of the asphalt concrete modulus to the PCC modulus. Reflection cracking would be reduced with a decreasing ratio. He concluded that there

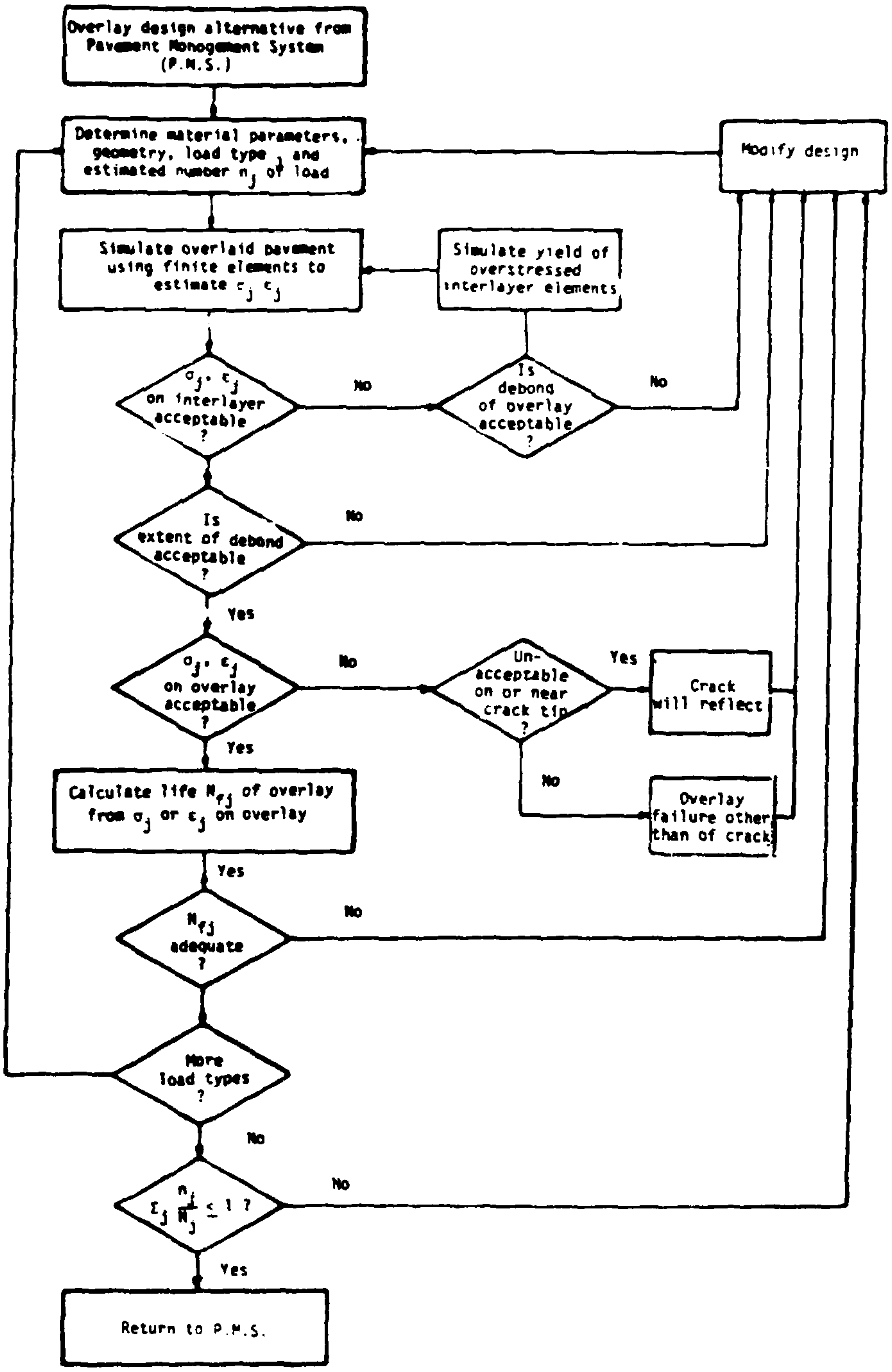


Figure 4. U.C. Berkeley Reflection Cracking Design Subsystem (after Reference 4.).

is no closed-form theoretical solution for calculating the potential for reflection cracking.

On the subject of general elastic analysis versus a fracture mechanics approach, Monismith and Coetzee (Reference 4) stated that stress intensity factor approaches may be useful in describing idealized reflection cracking due to thermal loads. This is because the resulting stresses occur normal to the crack plane in a tensile fashion. However, they also point out that these solutions assume a sharp tip when in actual practice the crack tip has a finite width. Also, since traffic loads occur in the crack plane, compressive stresses are present when the crack approaches the pavement surface. These cannot be properly accounted for in the current fracture mechanics methods.

<u>Ohio State University</u>

Majidzadeh, et al. (Reference 7) first attempted to apply fracture mechanics concepts to pavement systems in 1973. These concepts were considered for the analysis of fatigue failure of asphalt concrete. Suckarieh (Reference 6) extended the concept to flexible overlays of rigid pavements. He noted two phases in the overlay design process:

1. The evaluation of the existing pavement:
    a. Uniformity and quality of the PCC,
    b. Slab dimensions,
    c. Joint conditions,
    d. Type and condition of subgrade,
    e. Environmental effects, and
    f. Traffic.
2. The analysis of the system with the overlay:
    a. Joint movements due to traffic and
    b. Joint movements due to environment.

Suckarieh (Reference 6) developed a computer program called PLATES which uses finite element analysis to solve the case of a plate on elastic foundation. The mesh for the analysis is shown in Figure 5. He used the assumptions of linear stress and strain distributions in the slab and overlay as well as a perfect bond between the overlay and the slab. A flow diagram of the program is shown in Figure 6.

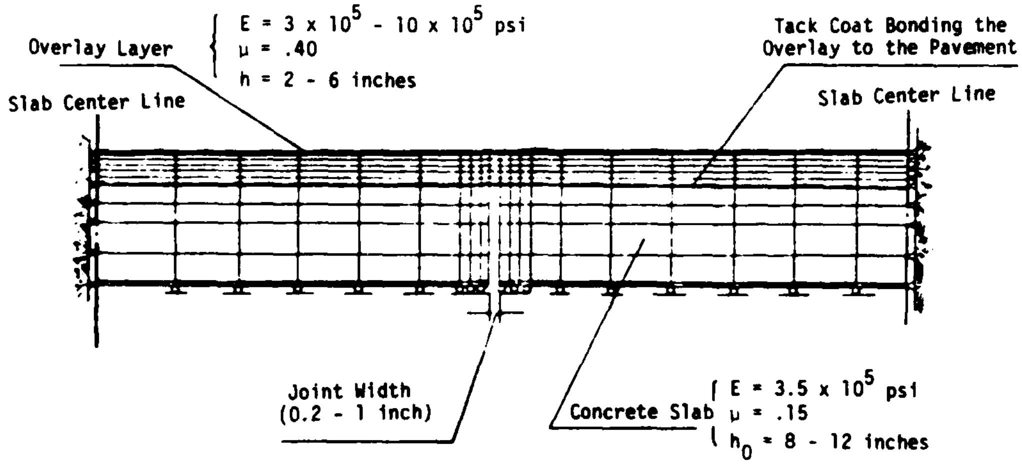


Figure 5. Finite Element Mesh Used in Ohio State University Stuey (after Reference 6).

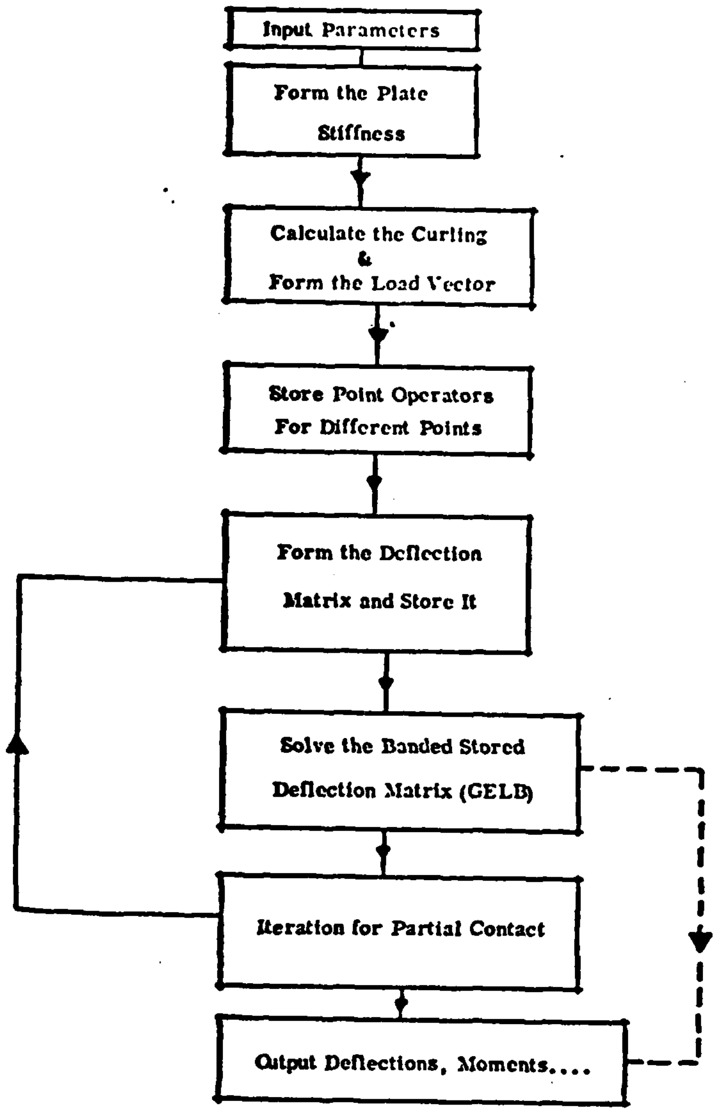


Figure 6. Flow Diagram of Ohio State University Reflection Cracking Analysis Program (Reference 6).

Suckarieh (Reference 6) assumed that cracks in asphalt overlays of PCC pavements start at the top of the asphalt concrete. The primary mechanisms of crack formation were the curling of slabs and horizontal movements which were the result of temperature changes. Typical shear and tensile stress distributions are shown in Figures 7 and 8, respectively. From these analyses, two nomographs were constructed to predict tensile stresses in the overlay (Figures 9 and 10).

To consider curling stresses, Suckarieh (Reference 6) assumed a situation such as that shown in Figure 11. The following relationship was used to calculate the radius of curvature in the overlay:

$$R = \frac{j}{2\theta} \qquad (7)$$

Where: R = overlay radius,
j = joint width and
θ = edge slopes.

Stresses in the overlay were then calculated by:

$$\sigma_{ov} = \theta \frac{E_{ov} h_{ov}}{j} \qquad (8)$$

Where: σ_{ov} = stress in overlay, and
h_{ov} = thickness of overlay.

A nomograph (Figure 12) was developed to determine the maximum stresses in the overlay from temperature differentials.

Monismith and Coetzee (Reference 4) contend that despite the bending condition shown in Figure 11, the overlay would probably be in tension throughout its thickness because:

1. Tensile stresses would develop due to the shrinkage of the restrained overlay and
2. The joint width would increase at the top of the PCC first followed by curling of the slab. This increased width can only result in tension in the overlay.

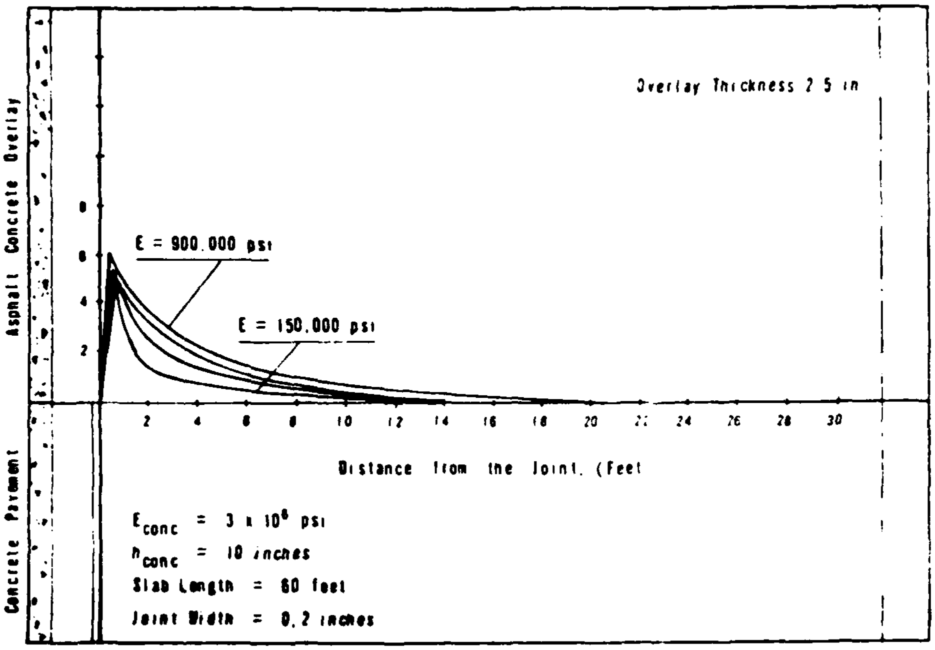


Figure 7. Shear Stress Distribution in Asphalt Concrete Overlay of PCC (after Reference 6).

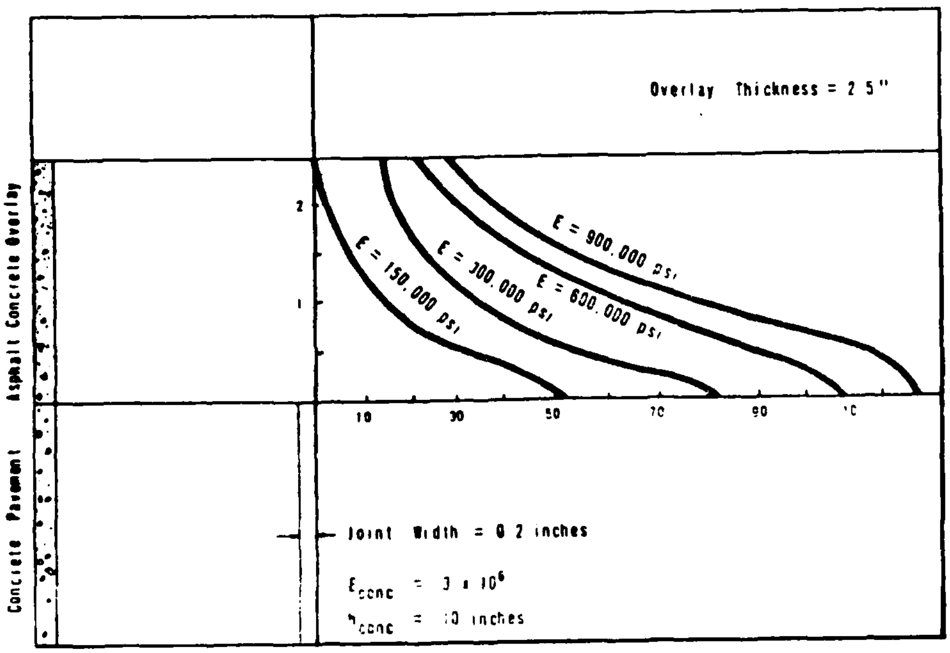


Figure 8. Tensile Stress Distribution in Asphalt Concrete Overlay of PCC (after Reference 6).

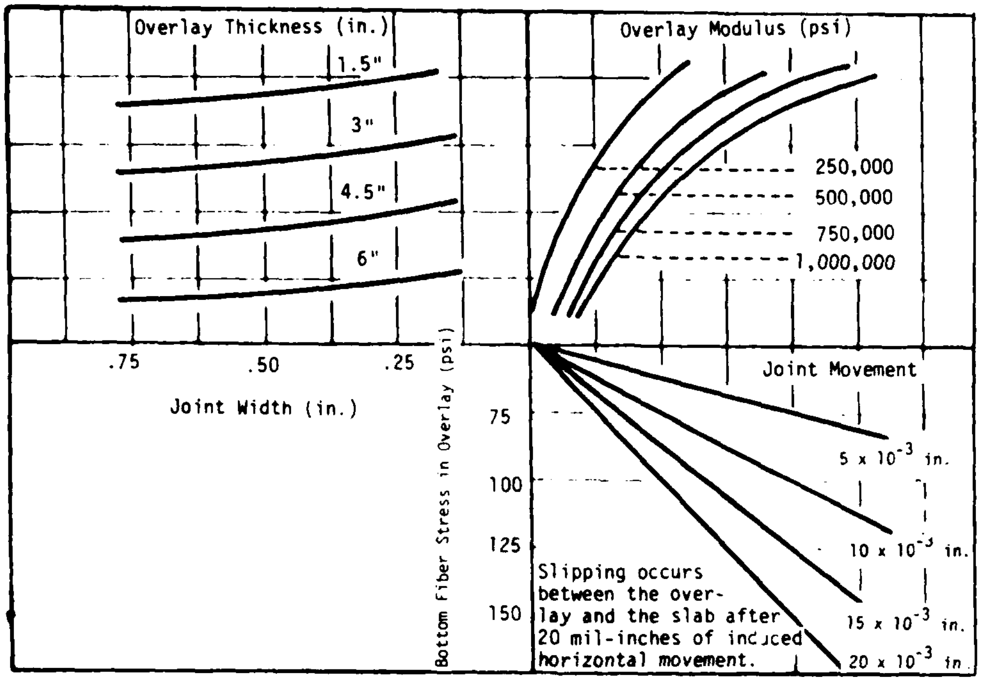


Figure 9. Nomograph to Determine Bottom Fiber Stress in Asphalt Concrete Overlay (after Reference 6).

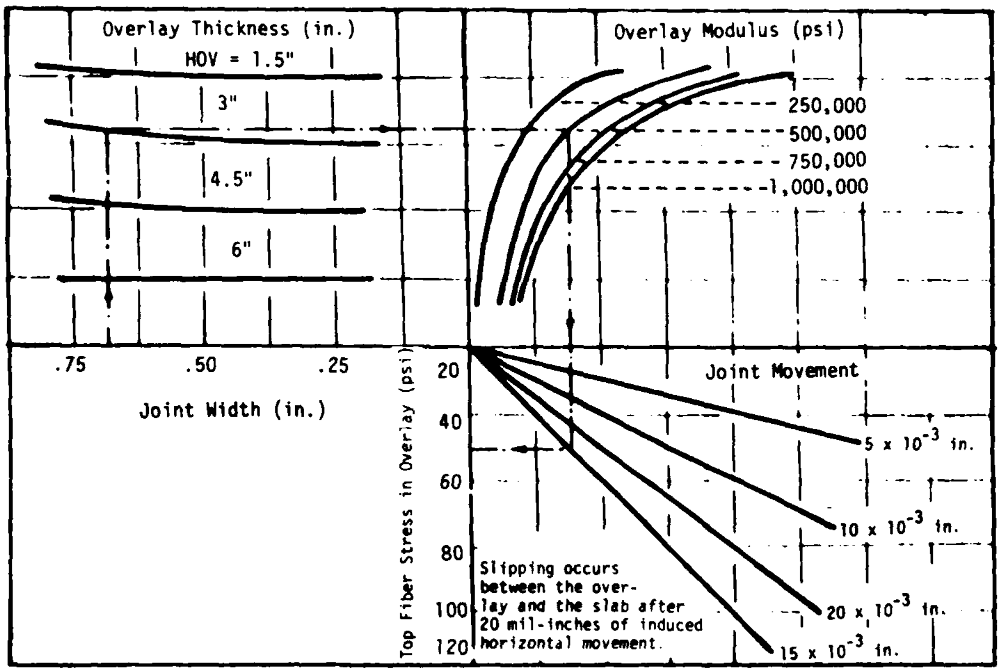


Figure 10. Nomograph to Determine Top Fiber Stress in Asphalt Concrete Overlay (after Reference 6).

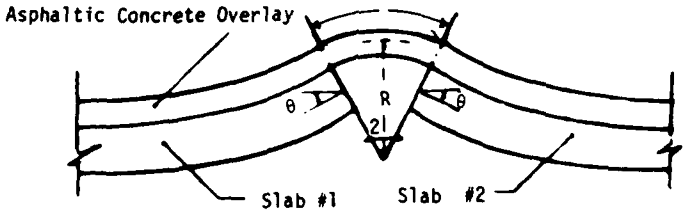


Figure 11. Bending of Overlay at PCC Joint (after Reference 6).

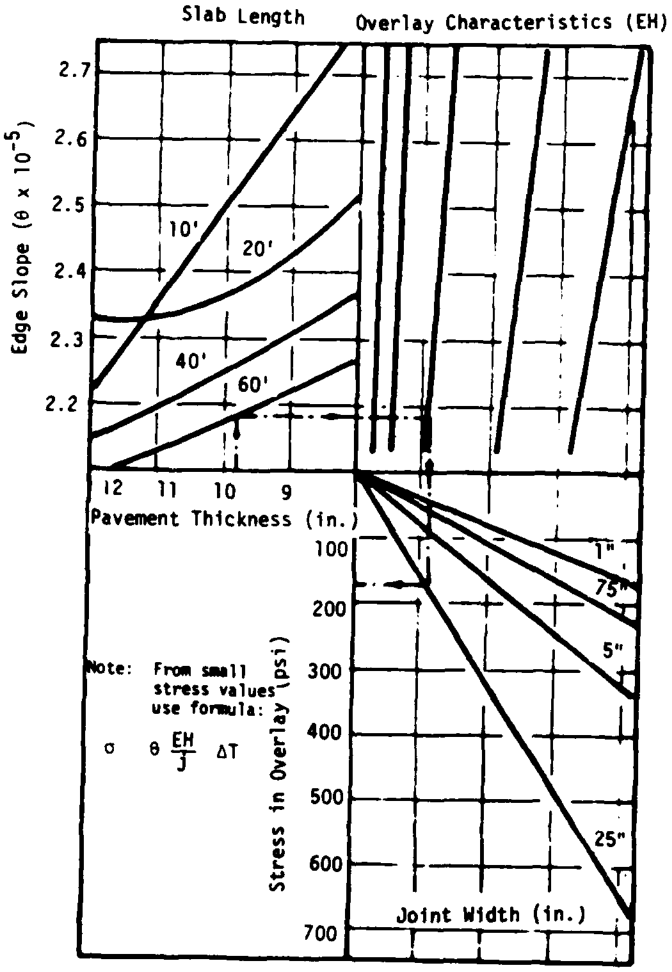


Figure 12. Nomograph to Determine Maximum Stress in Asphalt Overlay Due to 1°F Temperature Differential in PCC Slab (after Reference 6).

Texas A & M University

Elastic-Plastic Mechano-Lattice Analysis

Mechano-lattice analysis is a form of finite element analysis. Yandell and Lytton (Reference 5) used this type of stress-strain analysis for predicting reflection cracking. The technique may be used to investigate the effects of hysteresis in the prediction of pavement materials behavior. Materials which are nonlinear energy absorbing elastic or elastic-plastic may be used in conjunction with any value of Poisson's ratio.

Figure 13 shows that, for the analysis, the loading and unloading curves are simplified by straight lines. The three-dimensional unit analogy used in the analysis is shown in Figure 14. Each element was assigned different compliances for loading and unloading. The unit contains 28 elements connected at nodes. Stresses are calculated for the center of the unit by resolving the forces in the elements and dividing by the area of the side of the unit.

Lytton and Yandell (Reference 5) used a device called an "overlay tester" to estimate the material properties of the overlay. This test uses an asphalt concrete beam which has been halved transversely to simulate a cracked pavement. An intact layer of asphalt concrete or asphalt conrete plus interlayer is placed on the halved beam. Each beam half is attached to separate aluminum plates. One of the plates is connected to a servo-hydraulic ram which oscillates over a specified displacement at a specific frequency. Load is continuously monitored and the crack growth through the sample is measured after various cycles. This test simulates the horizontal movement which would be expected with volume change.

Lytton and Yandell (Reference 5) found that residual compressive stresses occur at the crack, upon closing, resulting in material being shoved into the area of the crack. This causes an increased thickness in this region. These humps would be compacted by traffic and could result in healing of cracked region. This may explain why reflection cracks seem to be more numerous in untrafficked areas of a pavement. As opening and closing of a crack proceeds, the tensile stresses near the crack tip decrease to a value lower than the compressive stress.

Viscoelastic Fracture Mechanics Analysis

Chang, et al. (Reference 9) presented a method to predict viscoelastic

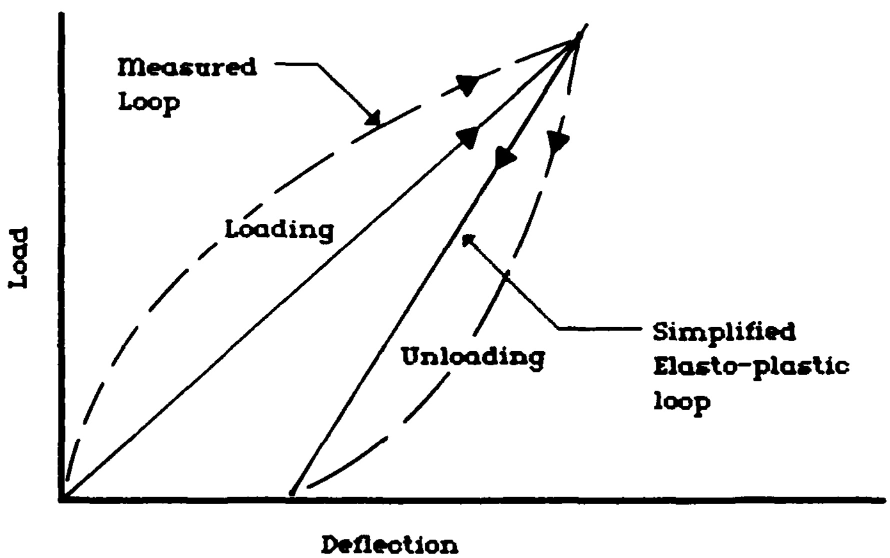


Figure 13. Elastic-Plastic Material Behavior
(Reference 5).

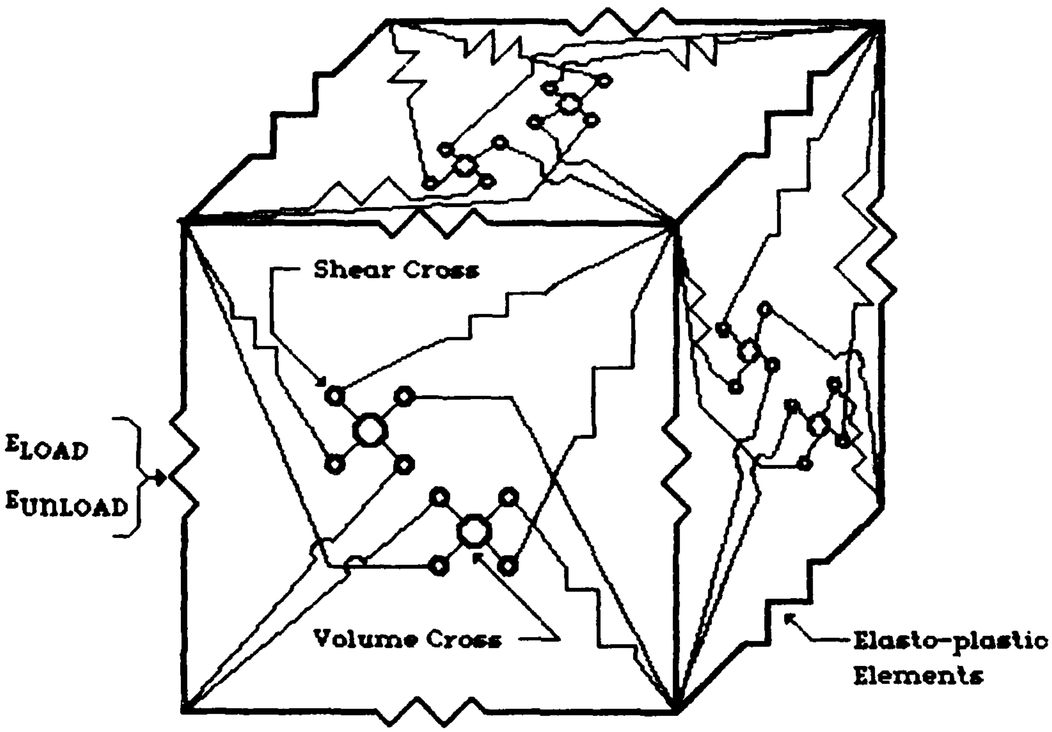


Figure 14. Mechano-Lattice Unit (Reference 5).

thermal stress in asphalt overlays of existing asphalt pavements. This entailed the use of a combination of linear elastic and viscoelastic stress analysis as well as viscoelastic fracture mechanics. The viscoelastic stress analysis was a modified procedure adapted from the the rocket propellant industry (Reference 56). The viscoelastic analysis was applied to the overlay and existing asphalt pavement. The thermal stress in the base course was treated in a linear elastic fashion.

Chang, et al. (Reference 9) considered crack growth in a pavement system to occur in three phases:

1. Crack tip is in the old pavement, propagating toward the overlay,
2. Crack tip is at the interface, and
3. Crack tip is in the overlay.

For each of these situations, a separate stress-intensity factor was calculated. For an overlaid PCC pavement, the first condition would already have occurred in the form of joints and existing cracks.

Germann and Lytton (Reference 10) used viscoelastic fracture mechanics in conjunction with the overlay tester to ascertain the effectiveness of fabric interlayers in retarding the growth of cracks in asphalt overlays. They found that crack growth was slower in pavement samples with fabric than in samples without fabric. They recommended a design procedure for determining the reflection cracking life of an asphalt overlay. This procedure uses viscoelastic fracture mechanics to determine the number of thermal cycles to which the overlay may be subjected without cracking.

This procedure was based upon the following expression:

$$\int_{1}^{N_f} dN = \int_{a_o}^{a_f} \frac{da}{2.14 \times 10^{-8} K^{4.63}} \qquad (9)$$

Where: N_f = number of cycles at failure,

a_f = final crack length (overlay thickness) and

K = stress intensity factor.

This integral may be solved using Simpson's Rule and substituting

$$\Delta N = \frac{\Delta a}{2.14 \times 10^{-8} K^{4.63}} \qquad (10)$$

To apply this equation in actual designs, assumptions must be made regarding the initial crack length, as well as parameters governing the value of K.

Austin Research Engineers (ARE)

Trebig, et al. (Reference 1) used conventional elastic analysis in their treatment of reflection cracking. They described the horizontal, vertical, and shear strains which act on an overlay in the following manner:

$$\varepsilon_H(T_i) = f[\varepsilon_{WL}\ (T_i) + \varepsilon_o(T_i)]_x \tag{11}$$

$$\varepsilon_v(T_i) = f[\varepsilon_{WL}\ (T_i)]_z \tag{12}$$

$$\gamma_{VH}(T_i) = \gamma_{HV}(T_i) = f\ [\gamma_{WL}(T_i) + \gamma_o(T_i)] \tag{13}$$

Where: $\varepsilon_H(T_i)$ = total horizontal strain parallel to the direction of traffic (x), at temperature T_i,

$\varepsilon_{WL}(T_i)$ = horizontal strain due to traffic load parallel to the direction traffic (x), at temperature T_i, calculated by layered elastic theory,

$\varepsilon_o(T_i)$ = horizontal strain due to movement of the concrete pavement as the result of temperature T_i, calculated according to the ARE model,

$\varepsilon_v(T_i)$ = total vertical strain in the overlay due to traffic load at temperature T_i,

$\gamma_{VH}(T_i)$ = total shear strain the overlay at temperature T_i,

$\gamma_{WL}(T_i)$ = shear strain caused by traffic load at temperature T_i, calculated by layered elastic theory and

$\gamma_o(T_i)$ = shear strain in the overlay due to traffic load and differential deflection across a joint or crack at temperature T_i, calculated according to ARE model.

Treybig, et al. (Reference 1) presented two methods of calculating the horizontal movement of PCC slabs due to temperature. One method assumed an unstabilized base with a frictionless interface between the PCC and the base. The other method was for a stabilized base and included a factor for

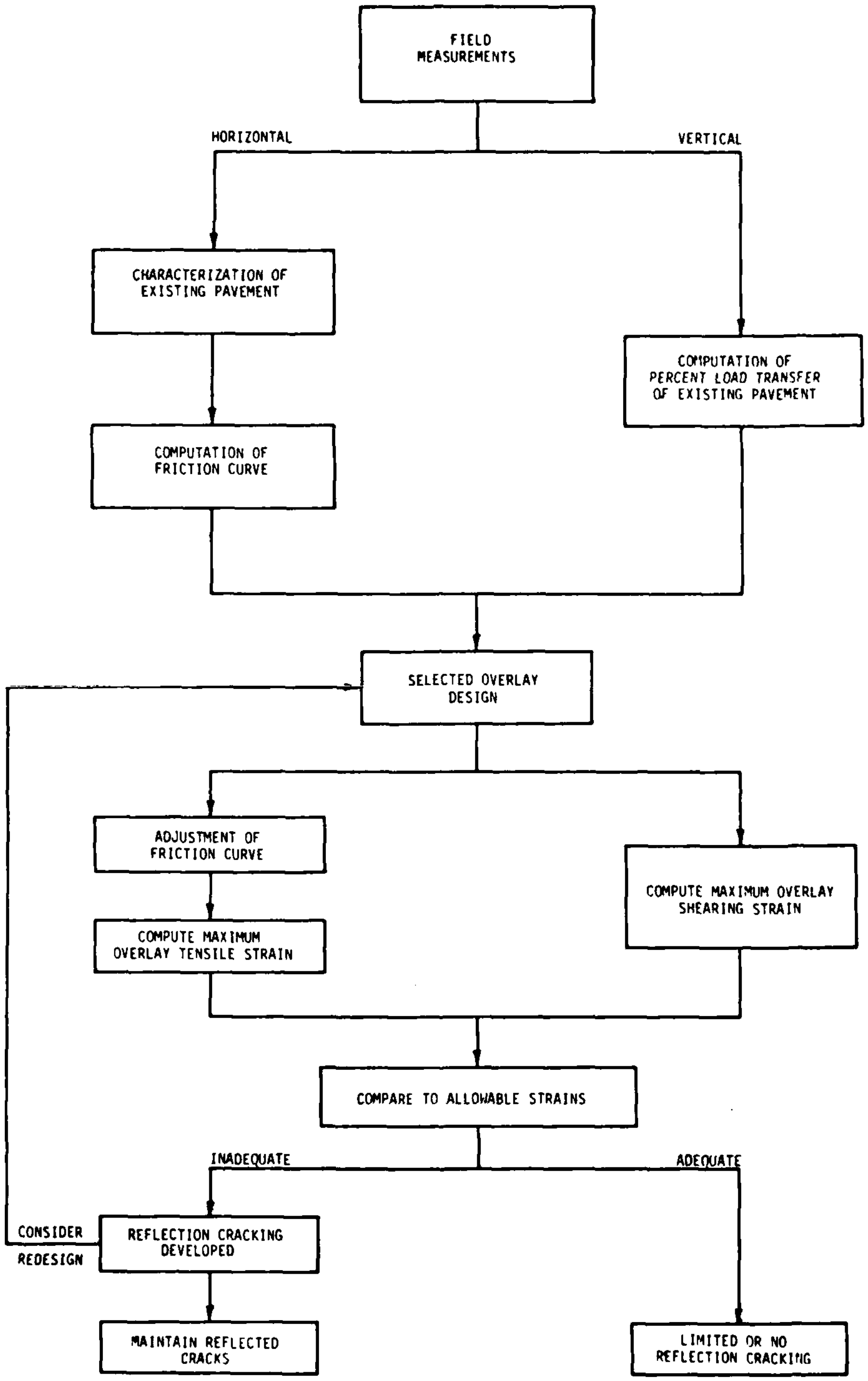


Figure 15. ARE Reflection Cracking Design Subsystem (Reference 1).

frictional resistance. The force at which sliding occurs is different for plain and reinforced concrete.

Horizontal strain occurring in the overlay was considered under two conditions: unbonded and bonded to the PCC. In the unbonded condition, these strains may be computed as:

$$\varepsilon_o = \alpha_o T_o \tag{14}$$

Where: α_o = thermal coefficient of the overlay material and

T_o = design temperature change of the overlay material for the overlay design life.

For the bonded case, the horizontal strain in the overlay may be computed as:

$$\varepsilon_o = \frac{F_{oc}}{A_o E_o} \tag{15}$$

Where: A_o = cross-sectional area of overlay per unit width of pavement,

E_o = creep modulus of the overlay material, and

F_{oc} = force in the overlay at a joint or crack.

The force in the overlay, F_{oc}, is dependent upon the creep compliance, cross-sectional area, and thermal characteristics of the overlay material; the movement of the concrete and the presence of a bond breaker.

The shear strain in the overlay material may be computed by the following formula:

$$\gamma_o = \frac{2\ \tau_o(1 + \mu_o)}{E_o} \tag{16}$$

Where: τ_o = shear stress in the overlay material as computed by a load transfer efficiency analysis,

μ_o = Poisson's ratio of the overlay material, and

E_o = dynamic modulus of the overlay material, assumed to be related to the shear modulus.

The design procedure developed by ARE (Reference 1) is outlined in Figure 15. ARE developed a computer program (Reference 57) to calculate

tensile and shear strains in pavement overlays according to the previously described methodology. The input for this program includes:

1. The existing pavement properties,
2. Characterization measurements,
3. Overlay properties, and
4. Other design input.

The results obtained from the program include:

1. Restraint coefficients,
2. Slope of the friction curve,
3. Stresses in the existing pavement, and
4. Overlay strains.

The following limitations and assumptions are applicable to the ARE method:

1. All assumptions inherent to linear elasticity,
2. Static equilibrium of the pavement,
3. Uniform distribution of temperature variation in the concrete pavement,
4. Concrete movement is continuous within the slab,
5. Movement of a layer is continuous through its thickness, and
6. Material properties are independent of space.

SECTION III

METHODS OF CONTROLLING REFLECTION CRACKING

GENERAL

Although the development of analysis methods for reflection cracking has not yet reached the point of implementation, methods for controlling reflection cracking have been used on a wide scale. These methods have all had variable success, although some seem to be more viable than others. Methods for controlling reflection cracking will be presented under five general categories:

1. Treatments of existing pavement,
2. The use of interlayers,
3. The use of cushion courses,
4. Application of thicker overlays, and
5. The use of special overlay materials and systems.

TREATMENTS OF EXISTING PAVEMENT

Breaking and Seating Slabs

It has been suggested that breaking and seating PCC slabs prior to an overlay with asphalt concrete is an effective means of reducing reflectic. cracking (References 11,31,47,58 through 62). Breaking and seating PCC slabs involves the use of equipment to exert large impact forces on the pavement to fracture it and heavy rollers (greater than 50 tons) to ensure full contact with the underlying pavement layer. Fracturing the slab into smaller dimensions reduces the amount of localized horizontal movement in the PCC, which results in lower tensile stresses being transmitted to the asphalt concrete overlay. Seating of the fractured PCC reduces the amount of vertical deflection caused by voids which may have formed at cracks or joints.

Lyon (Reference 59) reported that breaking and seating PCC slabs may reduce the amount of reflection cracking at joints by up to 50 percent (Figure 16). Noonan and McCullaugh (Reference 61) reported that in New York

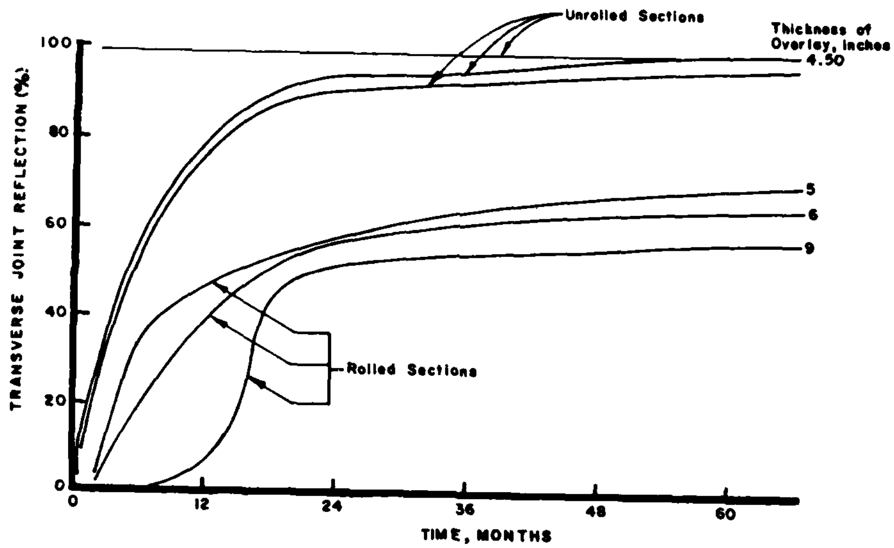


Figure 16. Effect of Breaking and Seating PCC Slabs on Reflection Cracking (Reference 59).

where 60- to 100-foot PCC joint spacings are used, breaking the PCC prior to overlay worked better than other methods of reducing reflection cracking. Brown and Voller (Reference 62) stated that this treatment used on an airfield runway at Fort Wainwright, Alaska resulted in cracking no more severe than would be expected from normal volume change in the asphalt concrete.

Others have also reported on the viability of breaking and seating PCC to control reflection cracking (References 63 through 66). The main drawback to this sort of treatment seems to be the expense associated with the time and equipment involved and loss of PCC structural capacity (Reference 61).

Subsealing

A variety of materials have been used for subsealing or undersealing PCC slabs at joints and cracks (References 11, 67 through 72). These include grout, asphalt, and lime. The purpose of subsealing is to increase the slab support and, in turn, reduce the amount of differential vertical movement between slabs. This is typically done by drilling holes in the PCC slabs in the vicinity of cracks and joints and pumping the subsealing material under the slab.

The State of Washington (Reference 73) uses a slightly different approach to subsealing. The method here involves injecting the subseal material horizontally from the side of the slab rather than vertically through the top. This method has the advantage of maintaining the integrity of the PCC. However, it may be of marginal use in airfield pavements where pavement widths are much greater than highways (especially PCC parking aprons).

INTERLAYERS

Interlayers are relatively thin membranes of low modulus materials placed over existing pavements or leveling courses prior to the placement of asphalt concrete overlays. There are two types of interlayers in general use: asphalt-rubber and fabrics. Monismith and Coetzee (Reference 4) postulated that there are two mechanisms by which interlayers may act to prevent reflection cracking:

1. A delamination may occur between the old and new pavement layers when the crack tip reaches the interface or

2. The strain energy at the crack tip may be dissipated by the use of an interface material capable of withstanding high strains.

They suggested that fabric interlayer systems behave in the manner of the first mechanism while asphalt-rubber systems are similar to the second mechanism.

Asphalt-Rubber

The U.S. Air Force sponsored research for the development of specification criteria for asphalt-rubber stress-absorbing membrane interlayers (SAMI) (Reference 13). This research encompassed extensive laboratory characterization of asphalt-rubber mixtures as well as the construction and subsequent monitoring of a field trial. The results of this study were guide specifications for asphalt-rubber SAMIs. However, without an understanding of the reflection cracking mechanisms, it was not possible to develop rational specifications.

Newcomb and McKeen (Reference 13) used a force-ductility (tensile) test which had been proposed by Anderson and Wiley (Reference 74). This test procedure showed promise in the development of rational material criteria for SAMI membrane evaluation. Of particular interest was the concept of a limiting strain criterion since strain at maximum stress was largely independent of other variables.

Way (References 75, 76, 77) found that asphalt-rubber membranes were one of the most effective treatments for reflection cracking. Vallerga, et al. (Reference 78) described surface preparation and construction techniques used in asphalt-rubber membranes constuction, as well as case histories of various jobs. Although numerous laboratory studies of asphalt-rubber studies have been conducted (References 13, 79 through 86), no nationally accepted standards have been developed. Also, the results of many of the field trials to date (References 13, 75, 87, 88, 89) have been inconclusive. Asphalt-rubber membrane construction is very sensitive to construction techniques and environmental conditions.

Fabrics

Asphalt-impregnated fabrics may be used to reduce the horizontal strain transferred from the PCC slabs to the overlay (Reference 1). However, due to the lack of shear strength in the fabrics, they may be of little value in redistributing shear stresses caused by differential vertical movement.

Dykes (Reference 90) suggested that design considerations for fabric interlayers include climate, drainage, and pavement conditions. He also stated that in cold regions, fabrics should be used primarily as waterproofing layers and not as reflection crack retarders. Eaton and Godfrey (Reference 91) seemed to confirm this assessment in their reflection cracking study at Thule Air Force Base, Greenland. They reported that after 1 year of service, overlays with fabric showed no less reflection cracking than overlays without fabric.

Preparation of the existing pavement prior to fabric installation may range from crack-filling to the placement of a leveling course. Dykes (Reference 90) emphasized that PCC slabs should be stabilized prior to fabric placement to minimize vertical movement at joints. Wrinkles will often appear in fabrics as they are being placed. These wrinkles should be remedied prior to overlay placement to prevent stress concentrations from occurring at these points. Dykes (Reference 90) suggested a minimum asphalt overlay thickness of 2 inches over PCC with a paving fabric interlayer.

Smith (Reference 92) documented 5 years of monitoring field trials incorporating three different interlayers fabrics over old PCC pavements in Iowa. He found that fabrics reduced transverse reflection cracking from 50 to 66 percent of that for sections without interlayers. A South Dakota report (Reference 93) states that fabrics have been used successfully to prevent moisture infiltration of frost-susceptible subgrades. Donnelly, et al. (Reference 94) concluded that nonwoven polypropylene fabric was the best interlayer system for the prevention of reflection cracking in asphalt overlays of flexible pavements. However, Mullen and Hader (Reference 95) stated that in an overlay study of an existing PCC pavement, fabric interlayers showed no better performance than other reflection cracking treatments. Noonan and McCullaugh (Reference 61) reported that on a PCC pavement with long joint spacing, fabrics did not perform well.

Dykes (Reference 90) identified the two most common causes of failure for fabric interlayers as local loss of wearing surface and fabric movement. He attributed the first problem to a lack of bond between thin overlays and the interlayers or to insufficient asphalt to satisfy both fabric and aggregate demand in chip seals. He stated that the second problem was due to traffic velocity and directional changes coupled with over-asphalting of the

interlayer. Fabric movement may also be caused by the use of temperature-susceptible asphalts, poor quality control of asphalt sealant distribution, or improper use of cutbacks and emulsions used as sealants. Thus, fabric performance seems to be largely dependent upon type and condition of existing pavement, construction techniques, environmental conditions during performance, and traffic characteristics.

Bond Breakers

Bond breakers is a term usually applied to interlayers which act as treatments of joints only. Materials such as sand, stone dust, wax paper, and fabrics have been used (References 1, 61, 88, 95). At best, these treatments have not shown any better performance than other methods of controlling reflection cracking.

CUSHION COURSES

A cushion course is a layer of untreated or asphalt-treated granular material placed on a PCC pavement prior to the placement of an overlay. These treatments usually have thicknesses greater than 1 inch (Reference 96). The following attributes have been identified for cushion courses (Reference 1):

1. Insulation of the PCC which reduces the thermal gradient and, subsequently, the horizontal movement of the slab,
2. Reduction of the tensile stain transmitted from the PCC to the asphalt concrete, and
3. Dampening of load-associated differential vertical deflection at joints and cracks.

Results of several studies (References 31, 47, 58, 65, 66, 96) have shown that cushion-course performance ranges from poor to excellent. Figure 17 illustrates the success of cushion courses in a Michigan study (Reference 95). Success has also been reported (References 97, 98,99) in the use of open-graded asphalt mixtures and macadams as cushion courses. In this system, a large amount of interconnecting voids (25 to 35 percent) allow the cushion course to absorb both vertical and horizontal slab movement.

Gradations for open-graded layers used in Tennessee and Arkansas are presented in Table 2. Hensley (Reference 99) recommended that 100 percent

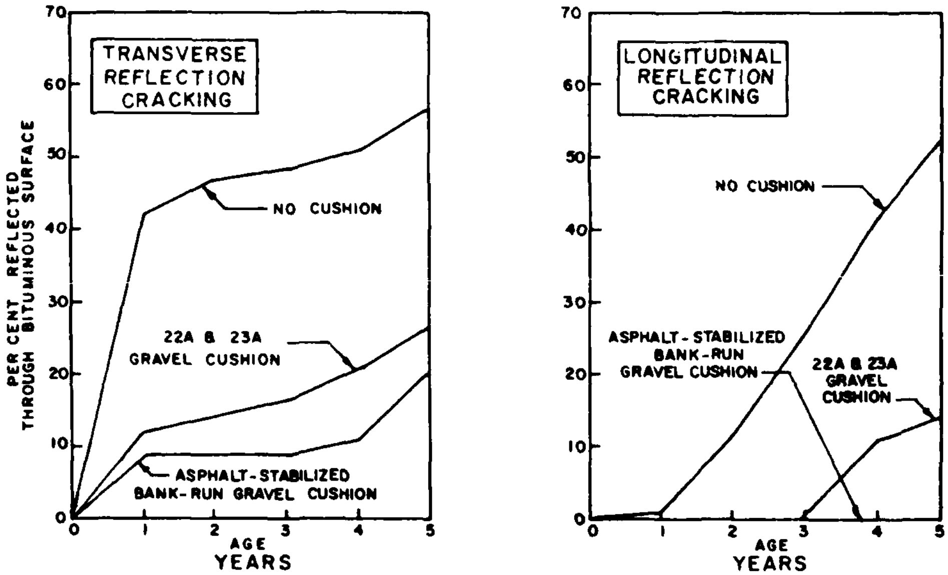


Figure 17. Effect of Cushion Courses on Reflection Cracking (Reference 95).

crushed, durable aggregates be used with a high-viscosity asphalt cement. He stated that a minimum of 1400 pounds (75-blow) Marshall stability or a minimum Hveem stability of 40 be used as a mixture criterion. For structural considerations, Hensley (Reference 99) recommended asphalt undersealing of the PCC and a minimum of 3.5-inch thickness for the cushion course. A typical road cross section is shown in Figure 18.

Treybig, et al. (Reference 1) noted that drainage problems may arise as a result of cushion courses. This is because the cushion course provides a channel through which water may flow, possibly causing stripping in the asphalt overlay or weakening of the cushion course. In a sensitivity analysis of unbound cushion courses, they found that:

1. The presence of a cushion course did not reduce the required overlay thickness for fatigue considerations and
2. Increasing overlay thicknesses are required for cushion courses of decreasing modulus.

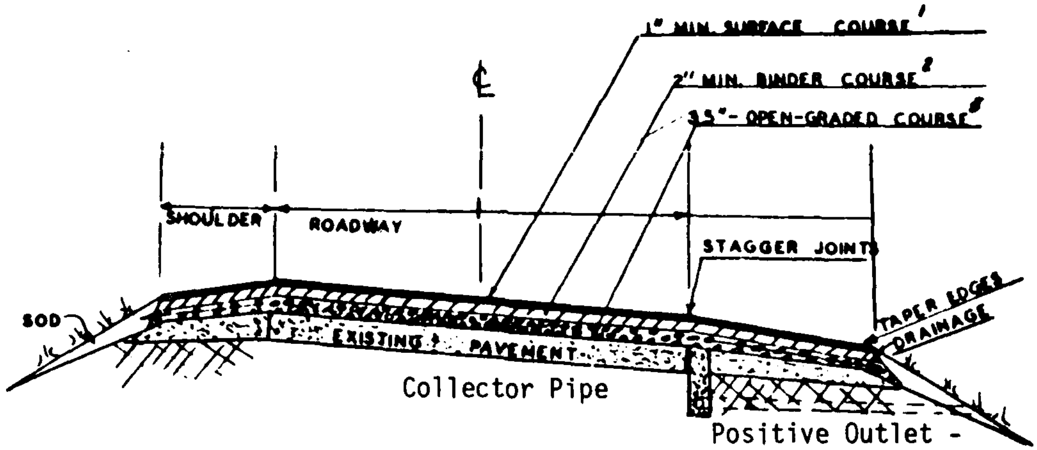
The first conclusion indicates that an unbound cushion course may be of marginal structural value. The second conclusion has economic and geometric implications.

THICKER OVERLAYS

The rationale for using a thicker overlay of asphalt concrete is that a thicker section will reduce stresses imposed by horizontal and vertical slab movements as well as reduce the temperature differential of the slab (References 31, 100). Treybig, et al. (Reference 1) attribute the success of thicker asphalt concrete overlays, primarily, to the greater resistance to movement provided by the thicker section, as well as the greater insulating effect. Others (References 63, 100, 101, 102) have shown that thicker sections of asphalt overlays will not eliminate reflection cracking but delay it at best.

Table 2. TENNESSEE AND ARKANSAS AGGREGATE GRADING SPECIFICATIONS FOR CRACK-RELIEF LAYER (AFTER REFERENCE 100).

Sieve	Percent Passing		
	A	B	C
3 in. (76 mm)	100	-	-
2-1/2 in. (64 mm)	95-100	100	-
2 in. (51 mm)	-	-	100-
1-1/2 in. (38.1 mm)	30-70	35-70	75-90
3/4 in. (19 mm)	3-20	5-20	50-70
3/8 in. (9.52 mm)	0-5	-	-
No. 4 (4.75 mm)	-	-	8-20
No. 8 (2-35 mm)	-	0-5	-
No. 100 (150 um)	-	-	0-5
No. 200 (75 um)	-	0-3	-
Asphalt Cement Content (AC-40) (AR-8000) (50-70) pen.		1.5-3.0%	



1. This surface course is a normal dense-graded mix.
2. The binder course is a normal intermediate course used for cover and leveling as required.
3. This open-graded mix, type A, B or C as listed in Table 1 is the crack-relief layer and should have 100% crushed particles.

Figure 18. Typical Road Cross-Section with Bound Cushion Course (after Reference 100).

SPECIAL OVERLAY MATERIALS AND SYSTEMS

Asphalt Specifications

It has been suggested (Reference 1) that special asphalt concrete overlay specifications might be used to preclude reflection cracking. This is done by designing the asphalt mixture to be more flexible. Tuckett, et al. (Reference 103) report that the use of different asphalt grades and admixtures have not been promising in reducing reflection cracking. Roberts (Reference 104) found that the use of soft asphalt grades was beneficial in allowing reflection cracks developed in the winter to mend during warm weather. A study conducted by the U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (Reference 91), concluded that the thickness of overlay was more important than the asphalt grade.

Research into the use of fillers such as limestone dust and asbestos fibers has shown that asphalt concrete stress-strain properties might be improved by these additives (References 11, 105, 106, 107). Because of recent research on their carcinogenic nature, asbestos fibers should probably not be considered for use in pavement construction. Natural rubber, reclaimed tire rubber, and neoprene have also been used in attempts to improve stress-strain characteristics of asphalt concrete overlays (References 47, 53, 76, 78, 108).

Special Overlay System

Steel reinforcement in asphalt concrete overlays has been reported (Reference 68) to redistibute the stresses caused by horizontal and vertical movement at joints and cracks. This is probably accomplished by disrupting the continuity of crack growth in the asphalt overlay. Expanded metal or welded wire mesh may be placed in strips covering only joints and cracks or laid over the entire pavement section. Numerous studies have been conducted to investigate the viability of steel reinforcement in asphalt overlays (References 30, 31, 47, 63, 68, 100, 109 through 112).

Although this method may be successful over the short-term, water may eventually infiltrate the asphalt concrete and cause the steel to rust. This would result in the formation of voids in the asphalt mixture and lead to rapid deterioration after the onset of corrosion.

DISCUSSION

A variety of methods have been used to prevent or retard reflection cracking in asphalt overlays. In all cases, it would be beneficial to ensure that the underlying PCC has firm support by either breaking and seating or subsealing. The use of interlayers would probably be must successful for areas where thermal cracking is not a major problem. However, even in cold regions, interlayers may reduce the amount of moisture infiltration to the subgrade. Cushion courses provide the desirable effects of tensile strain relief and thermal insulation but may cause drainage problems. Thicker overlays may delay but not necessarily stop or reduce reflection cracking. Special overlay treatments, designed to increase the flexibility of asphalt overlays, make sense, but inadequate performance data are available.

ARE (Reference 1) summarized treatments that have been used by various states to control reflection cracking (Table 3). Figure 19 shows a decision chart developed by ARE to assist in the determination of a preventative measure for reflection cracking.

TABLE 3. METHODS OF REFLECTION CRACK CONTROL TRIED IN VARIOUS STATES (REFERENCE 1).

Classification	Treatments to Existing PCC Pavements			Stress or Strain Relieving Interlayers			Cushion Courses	Special Overlay Treatments				Increase Overlay Thickness
								Reinforcement				
Location / Method	Crack Filling & Sealing	Breaking & Seating Pavement	Subsealing Joints /Cracks	Bond Breakers	Fabrics	Tack or Seal Coats		Steel	Fabrics	Asphalt Specifications	Additives	
Alabama						X	X					
Arizona	X	X		X	X	X	X		X	X	X	
Arkansas							X					
California	X			X	X	X		X		X	X	X
Colorado	X				X	X	X		X		X	
Connecticut				X				X			X	
Florida		X			X	X		X			X	X
Idaho							X	X			X	
Illinois					X		X	X	X			
Iowa	X			X				X	X	X		
Kansas					X							
Kentucky				X			X	X				
Louisiana	X	X			X							
Massachusetts								X		X	X	
Michigan							X	X	X			X
Minnesota		X			X			X				X
Mississippi	X				X							
Missouri		X			X		X	X			X	
Nevada							X					
New Jersey								X			X	
New Mexico	X						X					
New York		X	X	X				X				
N. Carolina			X		X							
N. Dakota					X	X		X			X	

TABLE 3. METHODS OF REFLECTION CRACK CONTROL TRIED IN VARIOUS STATES (REFERENCE 1). (CONCLUDED)

Classification	Treatments to Existing PCC Pavements			Stress or Strain Relieving Interlayers			Cushion Courses	Special Overlay Treatments				Increase Overlay Thick-ness
								Reinforcement				
Method / Location	Crack Filling & Sealing	Breaking & Seating Pavement	Subseal-ing Joints /Cracks	Bond Breaker	Fabrics	Tack or Seal Coats		Steel	Fabrics	Asphalt Specifi-cations	Addi-tives	
Oklahoma												
Oregon												
Pennsylvania												
S. Carolina												
S. Dakota												
Tennessee												
Texas												
Utah												
Vermont												
Virginia												
Washington												
Wisconsin												
Wyoming												

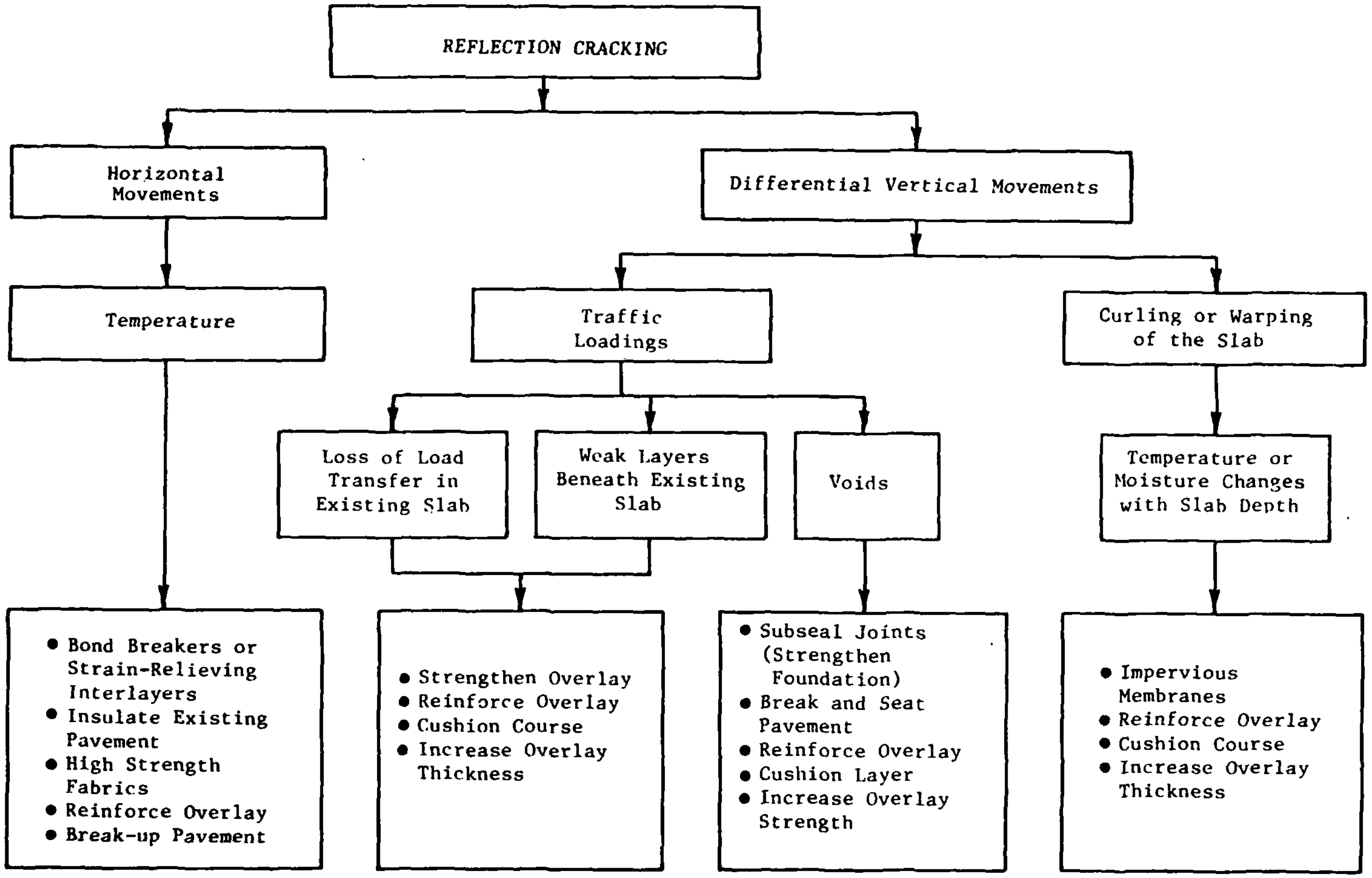


Figure 19. Flow Diagram to Determine an Appropriate Reflection Crack Control Method (Reference 1).

SECTION IV

TRIAL METHOD OF ANALYSIS USED IN THIS PROJECT

GENERAL CONCEPT

The main idea of the proposed methodology is to use in situ measured properties of PCC pavements to determine a required asphalt concrete overlay thickness to prevent reflection cracking for a desired design life. A conceptual outline of the method is shown in Figure 20. First, nondestructive testing (NDT) of the center and corner of the PCC slabs in a curled condition is accomplished, along with coring of the pavement to determine layer thicknesses. Next, layer moduli are estimated on the basis of center-of-slab measurements. An equivalent PCC modulus is calculated for the corner of the slab. This equivalent modulus is used in the estimation of traffic induced tensile strain in the overlay. Thermal strains in the overlay and PCC should be taken into account as a separate design subsystem. For a given thickness of overlay, the calculated strain should be compared to an allowable strain criterion. If the allowable strain is exceeded, the strain calculations are repeated, using increasing overlay thicknesses until the calculated strain is equal to or less than the allowable strain. This thickness would be the required overlay depth.

ESTIMATION OF LAYER MODULI

To approximate the layer moduli required for this procedure, it is necessary to have nondestructive test data at the slab centers, accurate section thickness measurements, and a means of calculating layer moduli from the center-of-the-slab NDT measurements.

Nondestructive Testing

The device used for nondestructive testing may be any device capable of exerting a sufficient dynamic load on the PCC pavement to produce and measure a deflection basin. In this study, a falling weight deflectometer (FWD) was used. FWD testing for this study was conducted by the Washington State

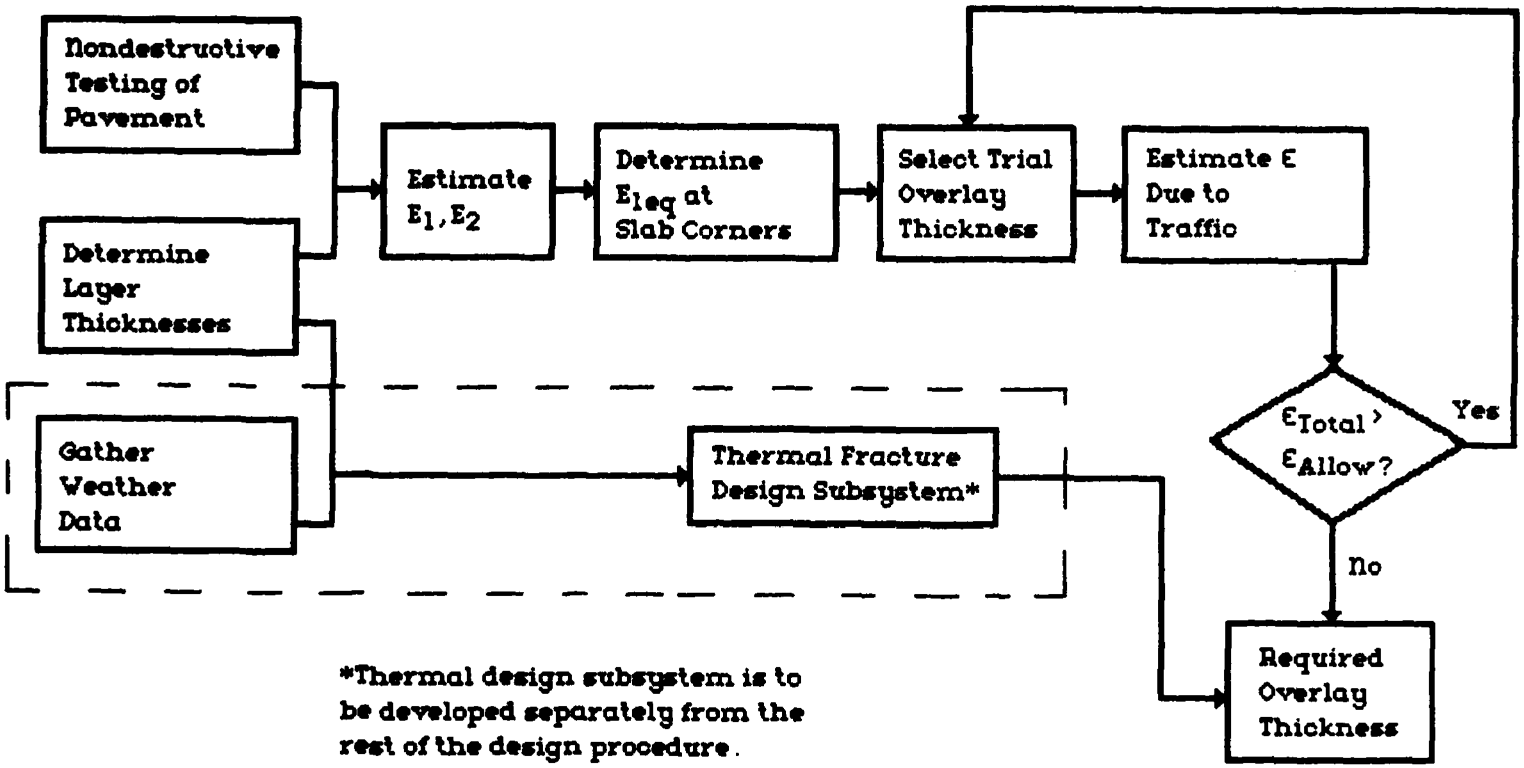


Figure 20. Conceptual Outline of Asphalt Concrete Overlay of PCC Design System.

Department of Transportation (WSDOT). The FWD was of the type described by Sorensen in Reference 113 and manufactured by Dynatest, Inc. of Ojai, California (Figure 21).

As implied by the name, the FWD force is exerted by dropping a weight from a known height. Geophones are spaced at known distances from the center of the load to measure pavement deflection. Figure 22 shows a sketch of a FWD measurement. Force exerted by the FWD may range from 1,500 to 24,000 pounds. The electronics of the system are such that the first peak deflection of each geophone is measured. These deflections are automatically printed out along with the force measurement on an on-board computer. Various falling weight deflectometers are described and compared to other NDT devices in References 113 through 116.

As shown in Figure 23, deflection measurements were taken at the center and corners of the slabs. The FWD testing for this study was conducted during early morning hours in the spring with the reasonable expectation that the PCC would be in a curled condition.

<u>Section Thickness</u>

It is important to obtain a good estimate of pavement layer thicknesses for this procedure. The next step (back-calculating elastic moduli) depends on layer thicknesses. For this study, thickness data were obtained from historical records at WSDOT, as well as physical coring of the pavements at two locations by WSDOT Materials Laboratory personnel. For Air Force pavements, consultation of past pavement evaluation reports may be adequate.

<u>Back-Calculating Elastic Moduli</u>

Because the FWD results are in the form of a deflection basin, it was necessary to use a computer program named BISDEF which was developed by Bush (Reference 117) to estimate layer moduli.

BISDEF provides a means for predicting the moduli of up to four layers from nondestructive deflection data by iteratively matching deflection values with material properties, using the BISAR layered-elastic program (Reference 118). A flow chart for BISDEF is shown in Figure 24. There may be a maximum of four deflection measurements and one load used for input. Deflection points are defined in terms of x and y coordinates, as well as depth. The load is defined in terms of its center x and y coordinates, vertical stress, and radius.

Figure 21. WSDOT Falling Weight Deflectometer.

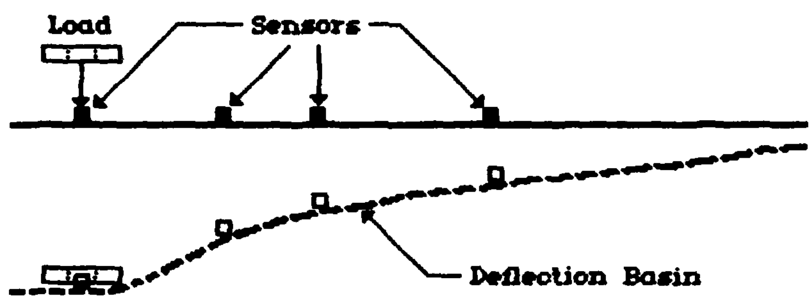


Figure 22. Sketch of FWD Deflection Measurement.

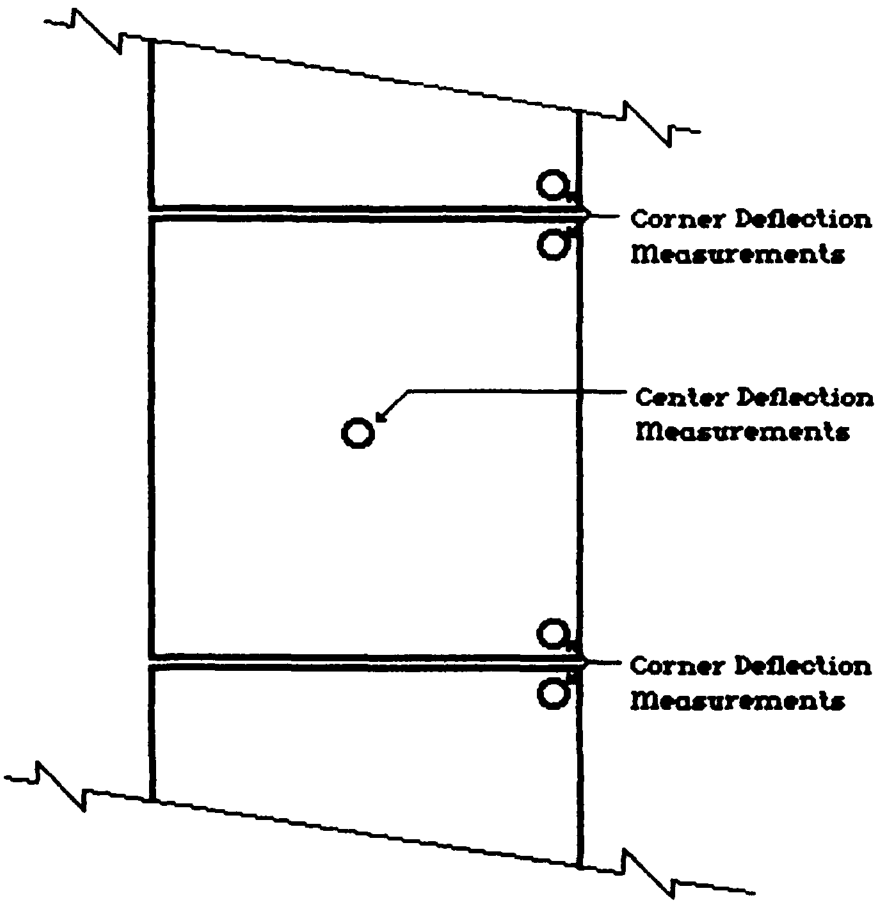


Figure 23. Diagram of Deflection Testing Scheme.

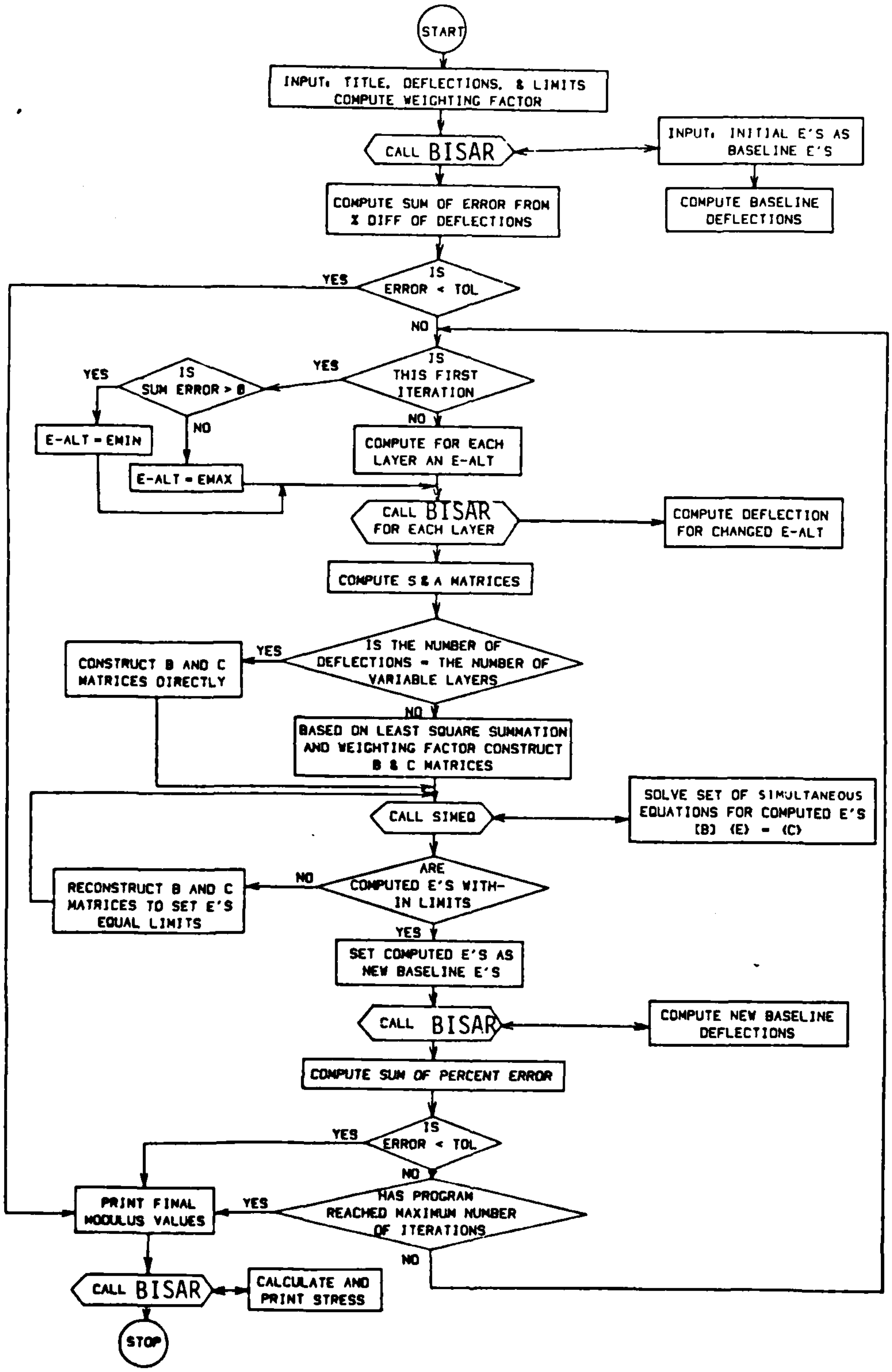


Figure 24. Flow Diagram of BISDEF (Reference 117).

A certain amount of judgment must be used when considering input values for initial material properties. Since the program iteratively matches deflection values with layer moduli, a tolerance must be specified for stopping the program. Ten percent is recommended for this value. Also, a maximum number of iterations (usually three) must be specified to stop the program to prevent the use of an excessive amount of computer time.

A minimum and maximum allowable modulus must be specified for each material of unknown modulus. Boundary conditions must be set as either rough or frictionless. An initial estimate of modulus, Poisson's ratio, and the thickness of all layers except the subgrade must be input to the program. The closer the initial modulus estimate is to the actual value, the faster the program will close and the less costly the run.

DETERMINING TRAFFIC INDUCED STRAIN

Equivalent PCC Modulus at Slab Corners

At this point in the analysis, the following information is necessary:

1. An estimation of moduli of the various pavement layers and
2. The maximum deflection at the slab corners during a curled condition.

By testing the concrete during a curled condition, two causes of reflection cracking are taken into account: vertical movements due to loss of subgrade support and warped edges caused by environmental conditions. For this analysis, the weakened corner is represented by a PCC layer of reduced modulus. The computer program, BISAR, is used to determine the reduced modulus.

BISAR uses elastic-layered theory to solve for stresses, strains, and displacements in pavement systems with one or more uniform circular loads applied vertically at the surface (Reference 118). BISAR has the capability of considering surface loads to be combinations of vertical normal and undirectional horizontal forces. The usual elastic-layered assumptions apply in this program except for continuity. Layer interfaces are assumed to either be in full continuity or frictionless.

To compute the reduced PCC corner modulus, the modulus values for the underlying layers are held constant in BISAR at values calculated for the

center of the slab. The corner PCC modulus is allowed to vary from the center-of-slab value (maximum) to one or two orders of magnitude lower. BISAR calculates the surface deflection on the basis of moduli used as input. Next, the equivalent corner PCC moduli are plotted against the computed maximum deflections as illustrated in Figure 25. The maximum measured deflection is found on the horizontal scale and a line is drawn to the curve. From the curve, the line is drawn to the vertical scale and the equivalent corner modulus is found. This equivalent modulus is then used to compute the traffic-induced tensile strain.

Fatigue Criteria

In most mechanistically based design procedures, fatigue failure is defined as a relationship between strain caused by a standard load and the number of repetitions of the load that the pavement will withstand before a certain level of cracking occurs. For airfield pavements, the standard load is usually expressed in terms of a particular type of aircraft, representing the airfield's mission. In highway pavement design, the standard load is often expressed in terms of an 18,000-pound (18-kip) axle. The 18-kip axle was the standard established by the AASHO Road Test (Reference 119). Since the pavements evaluated in this study were highways, the 18-kip standard was used.

Figure 26 shows established asphalt concrete fatigue relationships for the laboratory and field. The laboratory curve was developed by Monismith, et al. (Reference 120), for 3- by 3- by 15-inch beams subjected to repetitive bending. The field curve was developed by Finn, et al. (Reference 121), on the basis of the AASHO Road Test. The field curve represents the time to 10 percent fatigue cracking at various strain levels. The laboratory curve may be expressed as:

$$\log N_f = 14.82-3.291 \log (\varepsilon/10^{-6})-0.854 \log (E/10^3) \qquad (17)$$

and the field curve as:

$$\log N_f = 15.947-3.291 \log (\varepsilon/10^{-6})-0.854 \log (E/10^3) \qquad (18)$$

Where: N_f = Number of repetitions to failure,

ε = Initial strain for applied stress, and

E = Complex modulus.

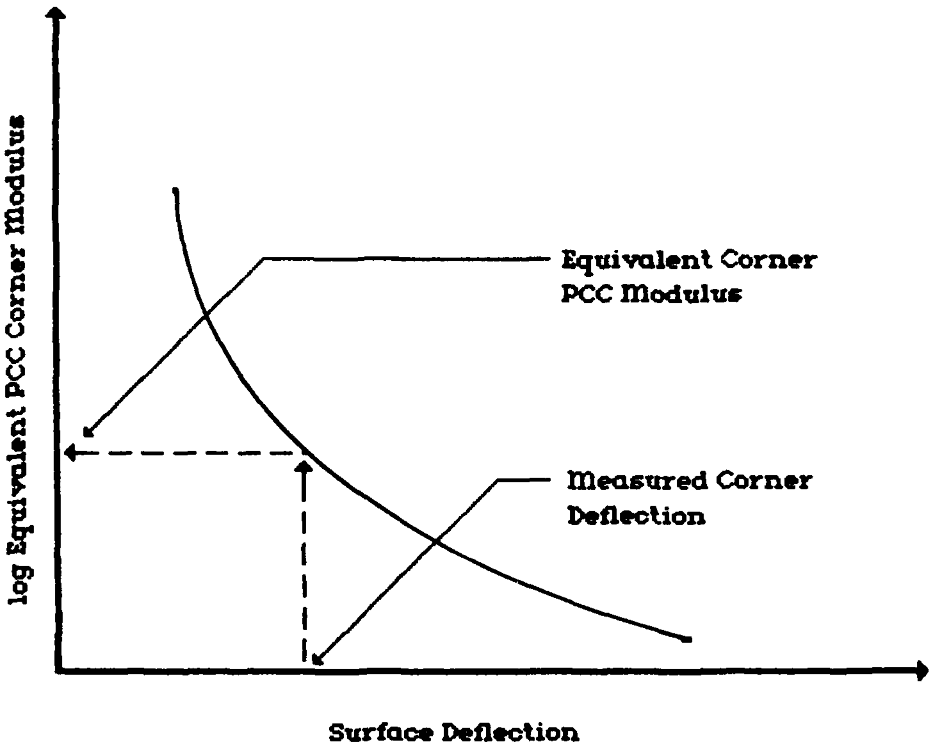


Figure 25. Method to Determine Equivalent Corner PCC Modulus.

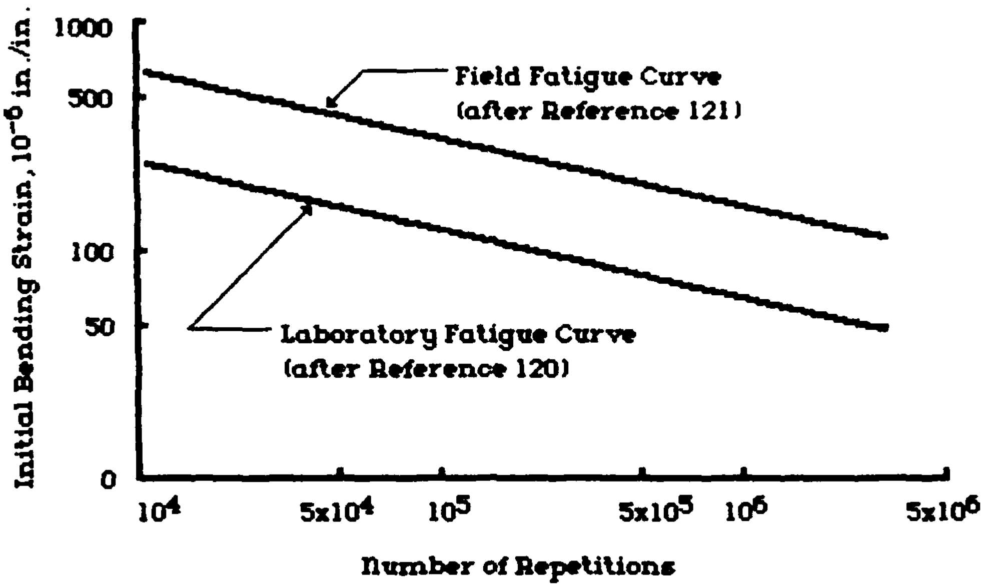


Figure 26. Laboratory and Field Fatigue Curves.

The tensile strain at the bottom of the overlay is calculated using the equivalent PCC modulus at the slab corner. This would be considered the critical loading condition, since it represents the weakest point of the pavement structure. The tensile strains computed, using layered elastic theory, were obtained with BISAR. Values of tensile strain may be compared to traffic repetitions to failure as shown in Figure 26.

ESTIMATION OF THERMAL STRAINS

Both the thermal strains in the asphalt concrete and in the PCC are acting upon the overlay. Thermal strain may be defined as:

$$\varepsilon = \alpha(\Delta T) \tag{19}$$

where ε = thermal strain ,

α = thermal coefficient, and

ΔT = temperature change

A typical value of thermal coefficent for asphalt concrete is 1.5×10^{-5} in/in/^{o}F (Reference 122) and a typical value for PCC is 5.0×10^{-6} in/in/^{o}F (Reference 46). Thus, asphalt concrete would have about three times as much thermal strain as PCC for an equivalent temperature change. If a full bond is assumed between the overlay and the slab, the concrete would restrain the movement of the bottom surface of the asphalt concrete and the critical thermal strain would occur at the top surface of the overlay. Therefore, thermal strains in asphalt concrete overlays should be accounted for in the same manner as in normal flexible pavements.

Thermal strains are normally considered separately from load-associated strains. This is because load-associated strains are treated on the basis of repetitions and thermal strains on the basis of a static or long-term condition. Methods of accounting for thermal strains in asphalt pavements range from specifying low-viscosity asphalt in cold climates to highly mechanistic procedures such as that proposed by Christison (Reference 123).

Thermal fracture criteria should be developed as a design subsystem which would be applicable to flexible overlays of rigid pavements, flexible overlays of flexible pavements, and new flexible pavement construction. The subsystem should be rationally based such that it could be applied globally

to match the Air Force mission. Temperature-dependent properties of stress, strain, and elastic modulus should be included, as well as climatic data.

LIMITATIONS AND ASSUMPTIONS OF THE DESIGN PROCEDURE

The following limitations and assumptions apply to the proposed design procedure:

1. The top surface of the pavement is assumed to have no shear.
2. The layers are assumed to have uniform thicknesses.
3. The layers are considered to be infinite in the horizontal plane. This assumption is very dependent upon the condition of load transfer mechanisms of the PCC slabs. This problem was not addressed in the proposed procedure and should be considered in future investigations.
4. Layer interfaces are considered continuous.
5. Layered elastic analysis may become cumbersome in instances where there is a thin, flexible layer over a thick, rigid layer.
6. Layered elastic analysis cannot easily accommodate thin interlayer systems.
7. The effects of a curled-up slab condition and support loss are considered collectively. The curled-up slab is a environmental condition and should perhaps be considered as such.
8. The condition of the bond between the existing PCC and the overlay can greatly influence the strain at the bottom of the overlay.

SECTION V

RESULTS AND DISCUSSION

TEST SECTIONS

Physical Descriptions

Test sections were selected for a preliminary trial of the proposed reflection cracking analysis procedure. Due to the exploratory nature of this study, highway pavements in Western Washington were selected to minimize the expense of the investigation. Overlay thicknesses on these pavements ranged from 1.8 to 4.2 inches over PCC slab thicknesses ranging from 6.6 to 9.5 inches. WSDOT personnel suggested that some slabs may have been as thick as 10.0 inches. Concrete joint spacings ranged from 15 to 20 feet. WSDOT has traditionally used plain transverse joints in PCC construction. The overlays were all placed in the mid to late 1970s.

State Route (SR) 162 had an overlay thickness of 1.8 inches over 9.5 inches of PCC. The PCC had straight joints spaced 20 feet apart. The overlay was placed in 1976 as part of a pavement widening project. A pavement survey in July 1984 showed that complete reflection cracking had occurred, but that the cracks were of low severity.

SR 603 is a rural farm road consisting of a 3-inch asphalt concrete overlay on 6.6 inches of portland cement concrete. A 15-foot PCC joint spacing was used with straight joints. The overlay was placed in September 1977, again as part of a pavement widening project. Complete reflection cracking was evident in July 1984. The cracks appeared to be of medium severity.

The test section at SR 5 Milepost (MP) 73.1 is a portion of Interstate 5. The road cross section is 4.2 inches of asphalt concrete over 9.5 inches of PCC. A 15-foot skewed joint spacing was used in the PCC construction. The overlay was constructed in March 1978. The PCC joints had been subsealed with asphalt cement prior to the overlay placement. The July 1984 survey showed the reflection cracks to be of medium severity with some spalling. It

was also noted that some slabs tended to rock with the passage of heavy trucks.

The test section located at SR 5 MP 85.3 is also a part of Interstate 5. The pavement cross section was a 4.2-inch overlay of 9.5 inches of PCC. A 15-foot joint spacing was used in conjunction with a skewed joint scheme in the PCC construction. The overlay was placed in May 1975. Low to medium severity reflection cracks were noted as well as midpanel cracking in four of the six slabs in the test section.

As with the other test sections located on Interstate 5, the test section SR 5 MP 86.7 consisted of 4.2 inches of asphalt concrete over 9.5 inches of portland cement concrete. Skewed joints at 15-foot intervals were used in the PCC placement. The overlay was constructed in July 1976. Low severity reflection cracks were evident in the overlay in July 1984. It was also noted that this test section was on a modestly high fill which could have made the subgrade seem relatively stiff.

Traffic

Traffic counts for the test sections were obtained from the Washington State Department of Transportation. These estimates of Average Daily Traffic (ADT) and the percent trucks for various years are presented in Table 4. The ADT ranged from 890 for SR 162 in 1978 to 33,400 for SR 5 MP 73.1 in 1983. Graphs showing ADT versus time and percent trucks versus time are found in Appendix A (Figures A-1 through A-10). The graphs also show when the pavements were overlayed and the estimated point at which reflection cracking started. The number of equivalent 18-kip axles between the time of overlay and time of cracking was computed according to the following formula:

$$N_f = (t)\left(\frac{ADT}{2}\right)(365 \text{ days/yr.}) \left(\frac{\% \text{ Trucks}}{100}\right)(F) \qquad (20)$$

Where: N_f = total number of equivalent 18-kip repetitions,

t = time between overlay and cracking, years,

ADT = average daily traffic in both directions, vehicles/day,

% Trucks = percent trucks in ADT, and

F = factor to convert the number of trucks equivalent to 18-kip axles.

The choice of F values came from a Texas study (Reference 124) which found that a value of 0.5 could be used for interstate highways and a value of 0.4 would be appropriate for rural highways.

TABLE 4. TRAFFIC HISTORY OF TEST SECTIONS.

Test Section	Year	ADT	Percent Trucks
SR 162	78	890	5
	83	1,300	11
SR 603	78	1,950	6
	83	2,100	9
SR 5 MP 73.1	76	22,400	-
	78	26,600	22
	83	33,400	26
SR 5 MP 85.3	76	24,404	-
	78	27,666	22
	83	28,800	21
SR 5 MP 86.7	76	24,404	-
	78	27,666	22
	83	28,800	21

Performance

The overall performance of the test sections is summarized in Table 5 along with descriptions of section thicknesses and joint spacings. The number of 18-kip axles between overlay and reflection crack initiation ranged between 1.07×10^4 and 3.20×10^6. The time for reflection cracking varied from about 1.5 to 5 years.

TABLE 5. SUMMARY OF PERFORMANCE FOR TEST SECTIONS.

Test Section	Layer Material	Layer Thickness, in.	Joint Spacing, Ft.	Date of Overlay	Date of Reflection Cracking	Number of Equivalent 18-kip Axles from Overlay to Reflection Cracking
SR 162	AC	1.8	20	76	81	1.87×10^4
	PCC	9.5				
SR 603	AC*	3.0	15	9-77	79	1.07×10^4
	PCC*	6.6				
SR 5 MP 73.1	AC*	4.2	15	3-78	83	3.20×10^6
	PCC*	9.5				
SR 5 MP 85.3	AC	4.2	15	5-75	79	1.90×10^6
	PCC	9.5				
SR 5 MP 86.7	AC	4.2	15	7-76	79	1.31×10^6
	PCC	10.0				

*Thicknesses confirmed with core measurements

FWD TESTING AND ANALYSIS

FWD Data

The results of the FWD testing are shown in Tables B-1 through B-5, Appendix B. The load is given with the associated deflections at distances of 0.00, 11.81, 25.59, and 47.24 in. for the centers and corners of the slabs. The lowest deflections were measured for test sections SR 5 MP 73.1 and MP 86.7. The highest deflections occurred at SR 162. Future studies should use higher loads with longer sensor spacings. The reason for this recommendation is the very low deflection values measured at the center of the slabs. These values are very close to being lower than the sensitivity of the equipment and a longer sensor spacing would give a better-defined deflection basin on stiff materials such as PCC.

Layer Moduli

Layer moduli calculated by BISDEF for each slab in each test section are presented in Tables C-1 through C-5, Appendix C. Large variabilities in layer moduli may be noted within a test section. There are two possible explanations for this. One is the nature of field data, i.e., construction variability and differing subgrade conditions within a test section. The other explanation is the range of modulus values given by BISDEF for a particular layer. Combinations of different initial layer moduli used in the program will give different final layer moduli which achieve the specified tolerance. This is a shortcoming of using layered elastic theory in cases of a thin, flexible layer over a thick, rigid layer. It indicates a need to develop a more appropriate model for reflection cracking.

Table C-6 presents the equivalent PCC modulus for each slab corner. These modulus values were obtained by holding the surface course and subgrade moduli constant and varying the PCC modulus in the BISAR computer program. The FWD maximum deflection at the slab corner was then matched to the surface deflection as shown in Figure 25. Equivalent PCC moduli ranged from 50,000 to 1,150,000 psi, indicating slab corner conditions ranging from very weak to very competent, i.e., marginal to firm slab-to-base contact.

DATA INTERPRETATION

The next step in the analysis was to evaluate strains at the bottom of the overlays at the PCC slab corners. This was done by using overlay and subgrade moduli calculated by BISDEF and the equivalent PCC modulus value determined in accordance with Figure 25 as material property. Since the FWD testing was conducted at temperatures close to 50°F for all the test sections, it was not considered necessary to adjust the asphalt concrete modulus values for temperature. A 9,000-pound, dual-wheel (18-kip axle) load was used for the loading condition. This strain was matched to a corresponding number of 18-kip axles.

The number of 18-kip axles between the time of overlay placement and the initiation of reflection cracking was computed as described eariler in this section. Not all of the reflection cracking occurred exactly at this point. Thus, knowing the traffic estimate at crack initiation and estimating the traffic to June 1984 (when all the reflection cracks were known to be visible), the traffic for each strain value was chosen as a random number between the two limits.

Strains were calculated in consideration of two different interface conditions between the overlay and the PCC slab corner. These were fully bonded and unbonded. The interface between the PCC slab corner and subgrade were fixed as being unbonded. Strains calculated for the bonded and unbonded cases with their corresponding random equivalent 18-kip numbers are presented in Appendix D.

<u>Results for Unbonded Overlay</u>

A graph of computed tensile strain versus the number of 18-kip axles is presented in Figure 27. Two curves are shown which indicate reflection cracking life increasing with decreasing tensile strain. The low curve is for all points on the graph and may be expressed as :

$$N_f = 9.3240 \times 10^{-11}\left(\frac{1}{\varepsilon}\right)^{4.006} \qquad (21)$$

Where: N_f = number of equivalent 18-kip axles and

ε = tensile strain at the bottom of overlay.

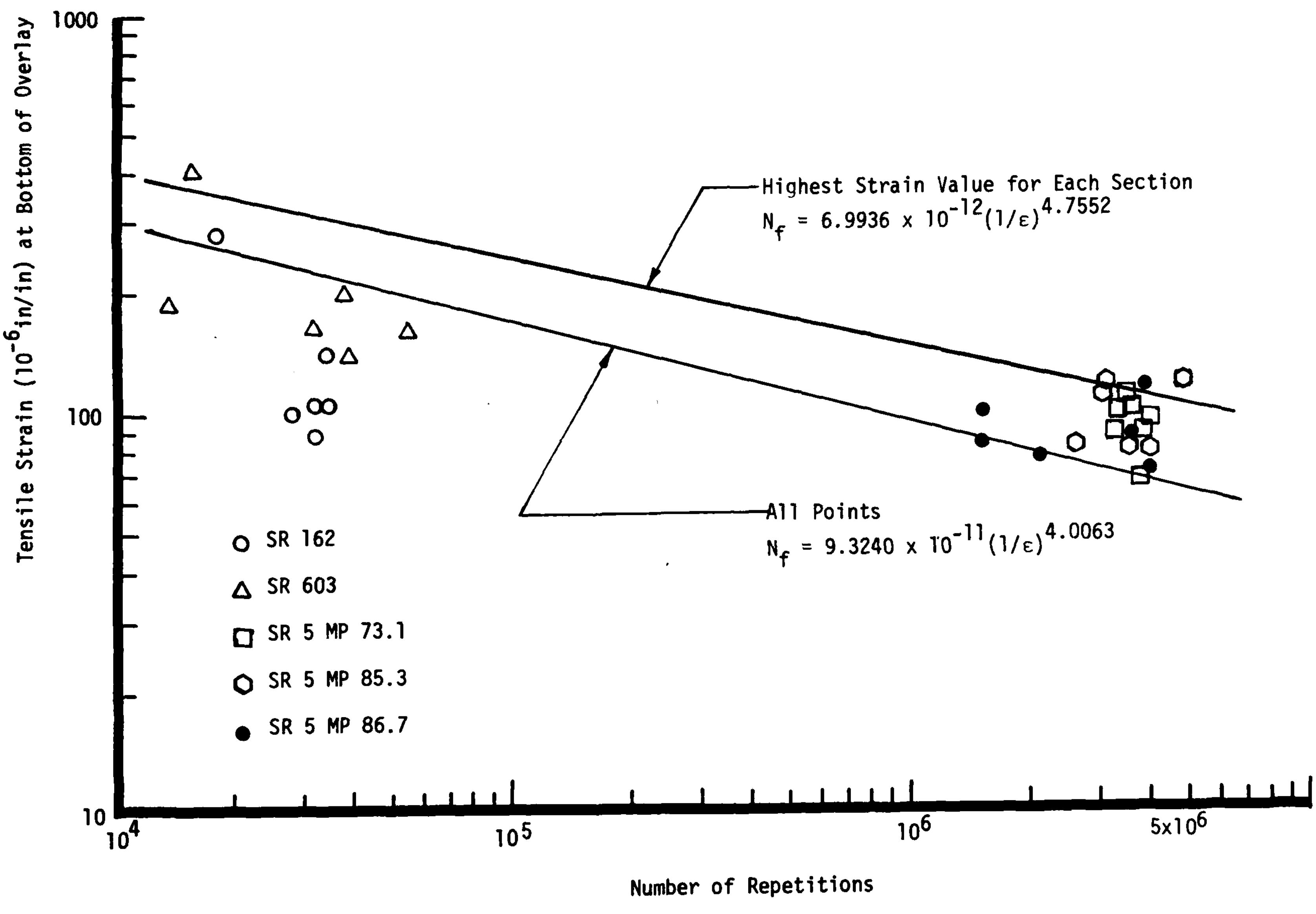


Figure 27. Tensile Strain versus Number of Repetitions for Test Sections - Unbonded Case.

The upper curve is shown to present the upper limit of an envelop. This curve was developed by performing a linear regression on the points of highest strain for each test section. It may be expressed as:

$$N_f = 6.9936 \times 10^{-12}\left(\frac{1}{\varepsilon}\right)^{4.755} \quad (22)$$

Results for Fully Bonded Overlay

Tensile strain versus number of repetitions assuming fully bonded overlays is shown in Figure 28. SR 162 was excluded from this analysis since much of that test section overlay was computed to be in compression instead of tension at the overlay bottom. This was due to the thin overlay thickness of 1.8 inches. The remaining points show a large variability. There is essentially no relation between all the points, but the top strain points do seem to decrease with increasing number of repetitions. The curve for the top points in Figure 28 is:

$$N_f = 2.1188 \times 10^{-12}\left(\frac{1}{\varepsilon}\right)^{4.470} \quad (23)$$

Comparison of Results with Other Studies

Figure 29 shows the reflection cracking initiation lines plotted with field (10 percent cracking) and laboratory fatigue curves obtained from References 121 and 120, respectively. The assumed modulus value for the field and laboratory curves was 1.6×10^6 psi. The curves obtained in this study are flatter than the curves from the other studies. The field and laboratory curves were for flexible pavements. The test sections would be more accurately described as composite pavements. Although the modulus values of the PCC joints are reduced in comparison to the slab center values, they are still much higher than what would be expected for a flexible base course. Thus, they could be expected to show a different relationship.

The curves from this study also show a definite difference in performance between bonded and unbonded overlays. Bonded overlays are subject to much less tensile strain than unbonded. The strain at the bottom of the bonded overlay is much more dependent upon the equivalent PCC modulus than the unbonded overlay. This would explain the increased data scatter noted in Figure 28.

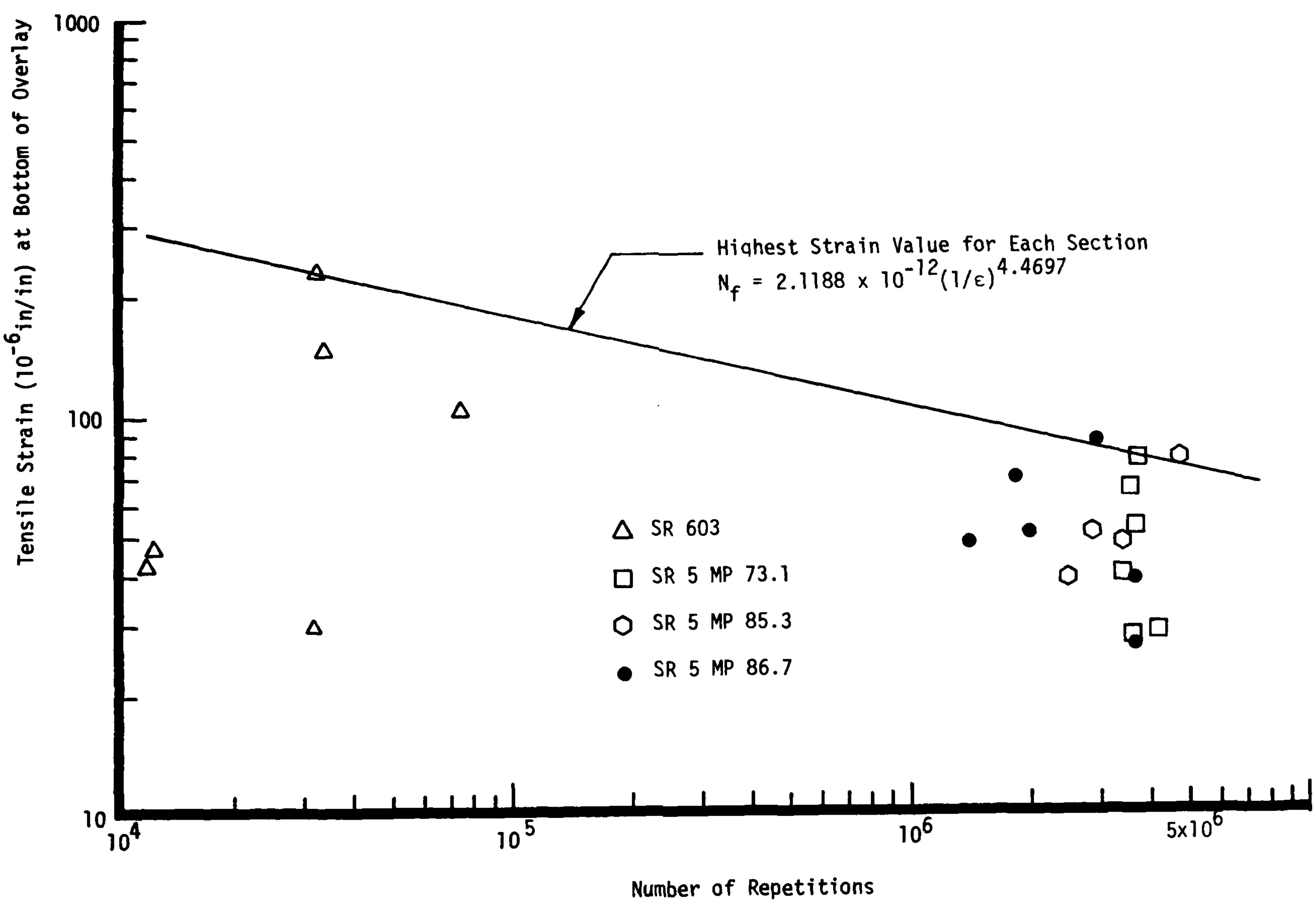


Figure 28. Tensile Strain versus Number of Repetitions for Test Sections - Fully Bonded Case.

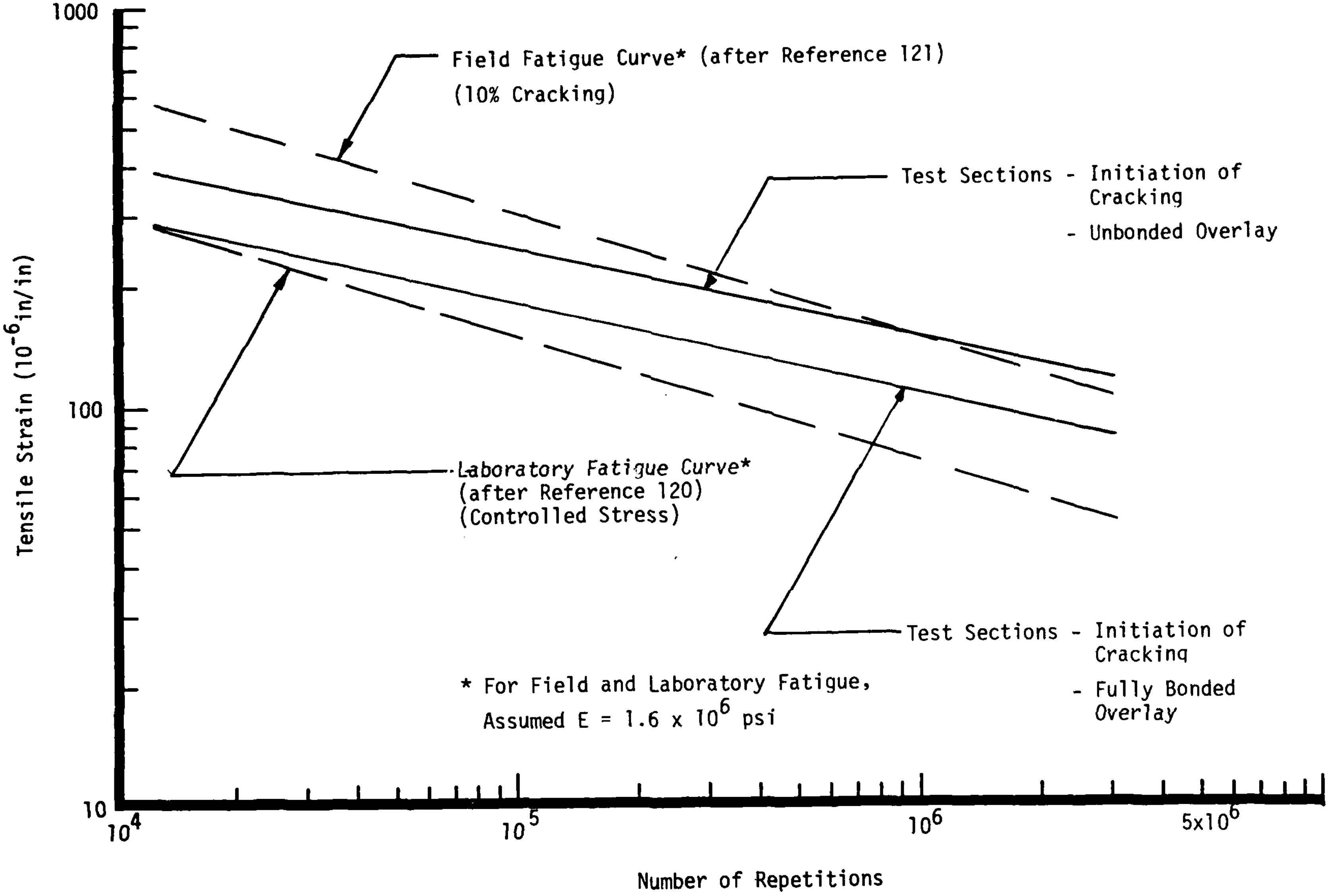


Figure 29. Comparison of Reflection Cracking Results to Established Fatigue Criteria.

EXAMPLE USE OF RESULTS

To illustrate how such relationships might be useful in pavement design, an example is included, using an 18-kip axle and fighter aircraft loading conditions. The loading conditions and traffic are specified in Table 6. All data in this example problem came from SR 5 MP 86.7. The pavement cross-section is 9.5 inches of PCC over subgrade. The results of FWD testing are shown in Table 7. From a BISDEF analysis of the deflection data, it was found that the PCC modulus was 5.19×10^6 psi and the subgrade modulus was 3.78×10^4 psi. The equivalent PCC modulus at the slab corner was found to be 1.7×10^5 psi, using the curve shown in Figure 30.

TABLE 6. EXAMPLE PROBLEM LOADING AND TRAFFIC CONDITIONS.

	18-kip Axle	Fighter Aircraft
Wheel configuration	Dual	Single
Wheel load, lbs.	4500	27,000
Tire pressure, psi	80	250
Design traffic, repetitions	1×10^6	1×10^5

TABLE 7. FWD RESULTS FOR EXAMPLE PROBLEM.

	Deflection at Distance, 10^{-3} in.			
Location	0.00 in.	11.81 in.	25.59 in.	47.24 in.
Slab Center	2.5	2.1	1.9	1.3
Slab Corner	6.3	-	-	-

Next, varying thicknesses of overlay are plotted against tensile strain (Figure 31) computed by BISAR using the equivalent PCC modulus. In this example, 50°F is the assumed design temperature with a corresponding asphalt concrete mixture stiffness of 1.6×10^6 psi. Curves based on both bonded and unbonded overlay conditions are presented. As would be expected, the unbonded overlay case would require a greater thickness than the bonded overlay for the same strain.

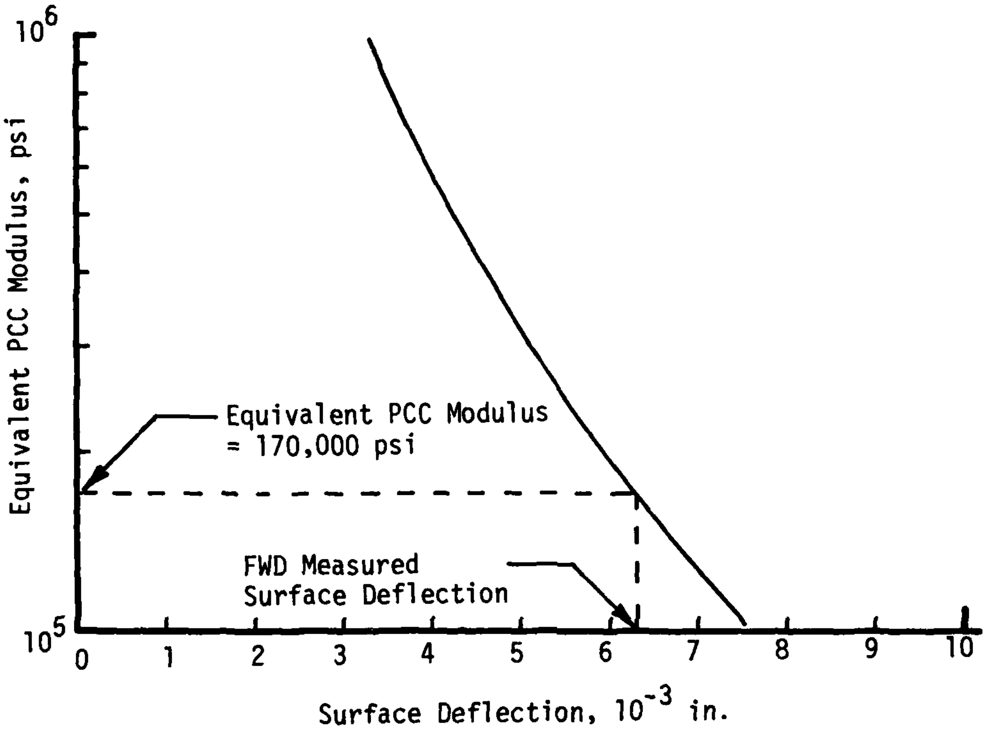


Figure 30. Equivalent PCC Modulus versus Surface Deflection for Example Problem.

Using the design traffic specified in Table 6, Figures 27 and 28 are consulted to obtain the overlay tensile strain for the unbonded and bonded case, respectively. Assuming that no reflection cracking is desired, the top curve is used in Figure 27. By entering the strain values obtained from Figures 27 and 28 on the strain axis in Figure 31, the minimum overlay thickness may be determined. The results are summarized in Table 8. If a minimum allowable thickness of 2 inches is assumed, this would be the thickness required for the 18-kip axle for the bonded overlay. The 18-kip axle with an unbonded overlay requires 3 inches of asphalt concrete. The required thicknesses for the unbonded and bonded overlays for the fighter aircraft would be 6 inches and 5 inches, respectively.

TABLE 8. SUMMARY OF RESULTS FOR EXAMPLE PROBLEM.

1. Fighter Aircraft, $N_f = 1 \times 10^5$
   a. Unbonded Overlay

      $= 255 \times 10^{-6}$ in/in (Figure 27)

      Required Overlay Thickness = 6 inches

   b. Bonded Overlay

      $= 185 \times 10^{-6}$ in/in (Figure 28)

      Required Overlay Thickness = 5 inches

2. 18-kip axle, $N_f = 1 \times 10^6$
   a. Unbonded Overlay

      $= 110 \times 10^{-6}$ in/in (Figure 27)

      Required Overlay Thickness = 3 inches

   b. Bonded Overlay

      $= 78 \times 10^{-6}$ in/in (Figure 28)

      Required Overlay Thickness = 2 inches

      (assuming a minimum allowable thickness of 2 inches)

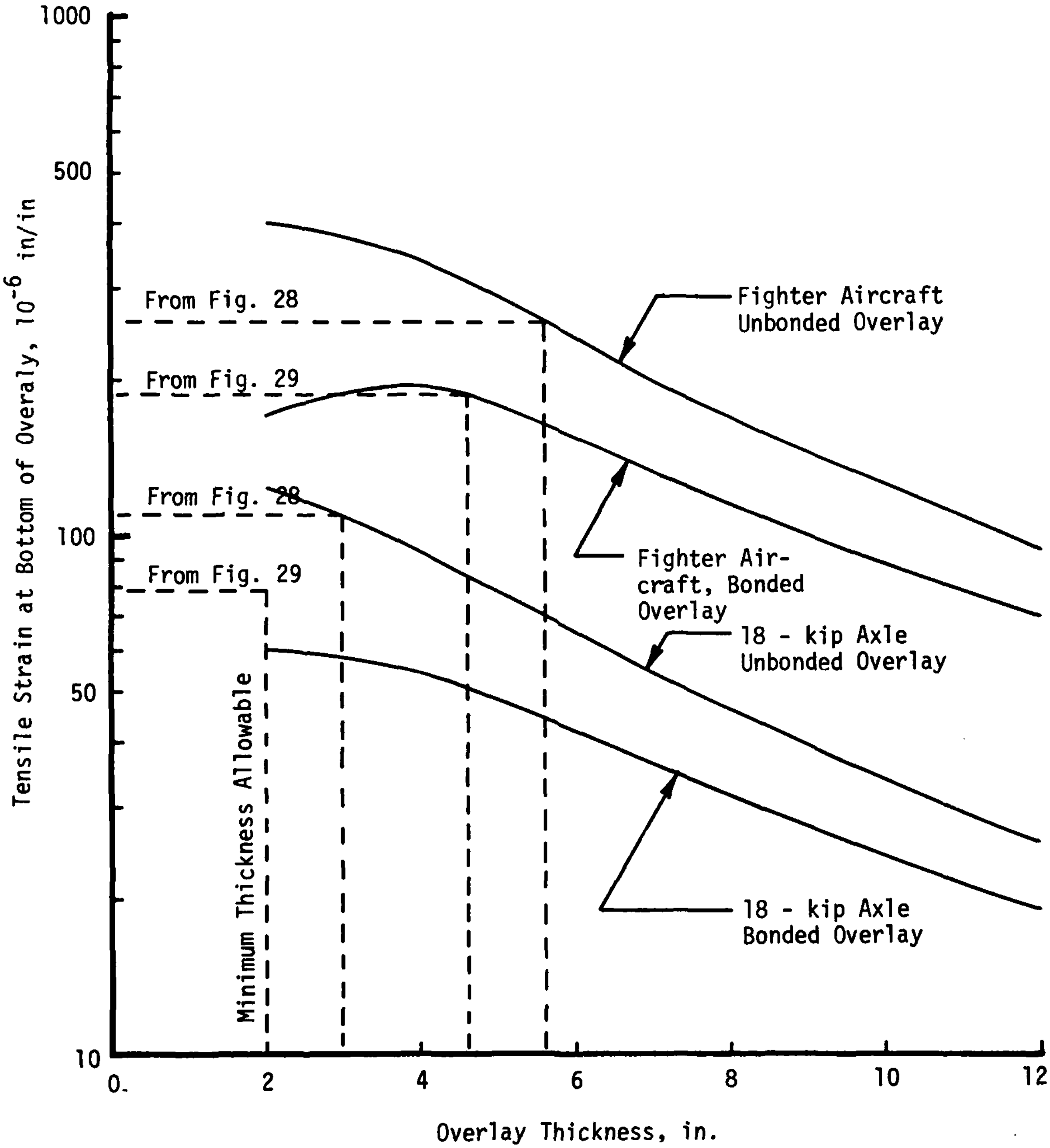


Figure 31. Tensile Strain versus Overlay Thickness for Example Problem.

SECTION VI

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The following conclusions are based upon the results of this study.

1. A variety of mechanisms may act together to cause reflection cracking in asphalt concrete overlays of portland cement concrete.

2. There is little agreement among researchers on how these mechanisms act in causing reflection cracking. A variety of models are offered ranging from simple elastic to complex fracture mechanics treatments of reflection cracking.

3. Many approaches have been tried in reducing reflection cracking, but none have been shown to be universally successful.

4. Reducing the vertical movement of the PCC slab edges by subsealing or breaking and seating are possibly two of the most effective means of reducing reflection cracking. However, the breaking and seating method is detrimental to the structural integrity of the system.

5. A new method was proposed for evaluating the potential of reflection cracking in asphalt concrete overlays of PCC based upon NDT data.

6. Based upon a limited analysis, reflection cracking may be related to tensile strain developed in the bottom of the overlay and the magnitude and number of repetitions of load.

7. Layered elastic analysis may be of marginal value in evaluating systems having thick, rigid layers underlying thin, flexible layers.

8. The condition of the interface between the portland cement concrete and asphalt concrete is important in the attempted analysis procedure.

RECOMMENDATIONS

This study has raised more questions than it answered. The following recommendations are made on the basis of the findings:

1. More work needs to be directed at defining the mechanisms of

reflection cracking. Specifically the following questions should be addressed:

a. What is the impact of PCC curling stresses on asphalt concrete overlays?

b. What is the impact of traffic loading on overlays for curled and uncurled PCC slab conditions?

c. What part does load transfer play in reflection cracking? If shear stress is found to be significant, then what criteria should be used to define allowable shear stress?

2. A model should be developed which accounts for the defined mechanisms. It is suggested that the model be one which does not rely on layered elastic theory.

3. The basic approach of using NDT data to evaluate potential reflection cracking should be followed. In this manner, field data may be used to solve field problems.

4. The pavements tested in this study were all located in Western Washington. Many of the construction and environmental conditions were the same for the test sections. A broader testing scheme is needed to extend the data base, including thicker PCC pavements.

5. The basic premise of the proposed method of analysis was that the maximum tensile strain in the overlay occurred at the bottom. This point has been both contended and supported by other researchers. A fundamental study is needed to resolve this issue.

6. One method which could be used to gain valuable insight on the causes and remedies of reflection cracking is to survey pavement engineers with practical experience in these matters. With this in mind, a questionnaire has been prepared for Air Force pavement engineers and is presented in Appendix E. The intent of the proposed questionnaire is to identify Air Force facilities with overlayed PCC pavements. Following this identification, personal followups would be used to obtain needed details.

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APPENDIX A

TRAFFIC TRENDS FOR TEST SECTIONS

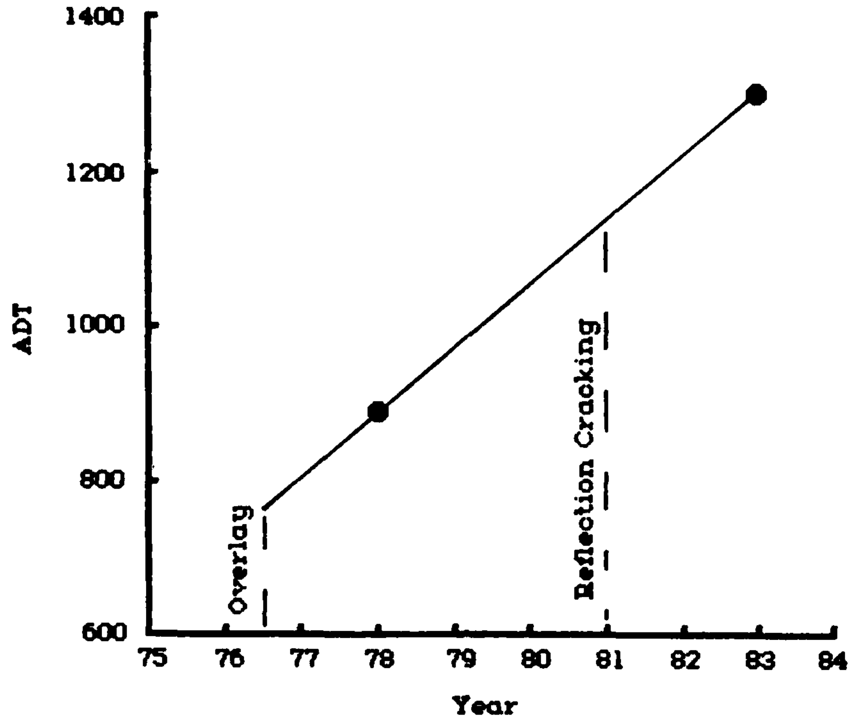
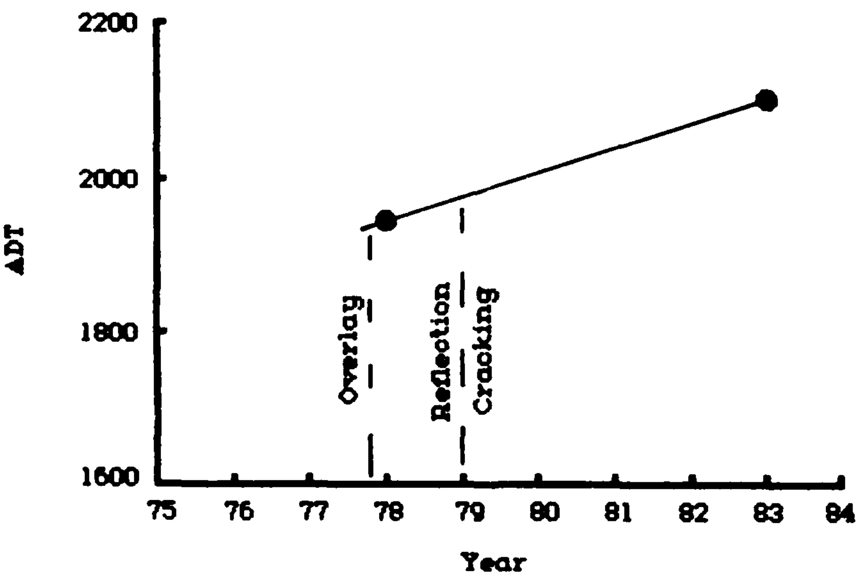


Figure A-1. ADT versus Time for SR 162.



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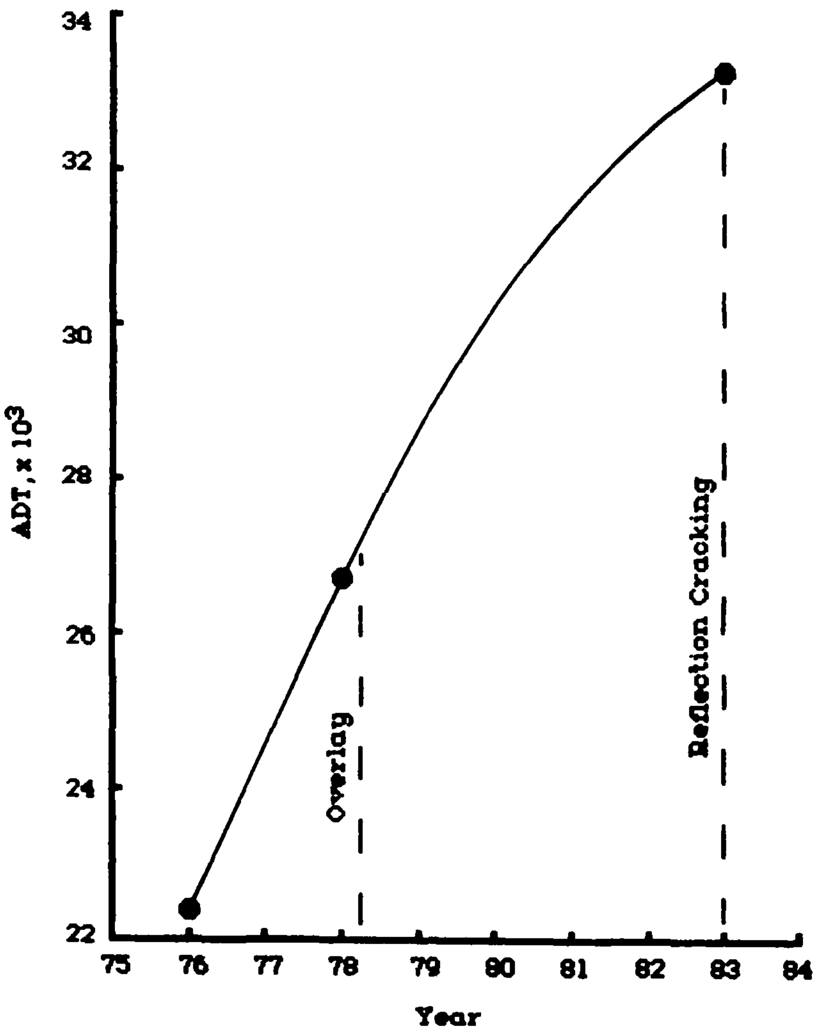


Figure A-3. ADT versus Time for SR 5 MP 73.1.

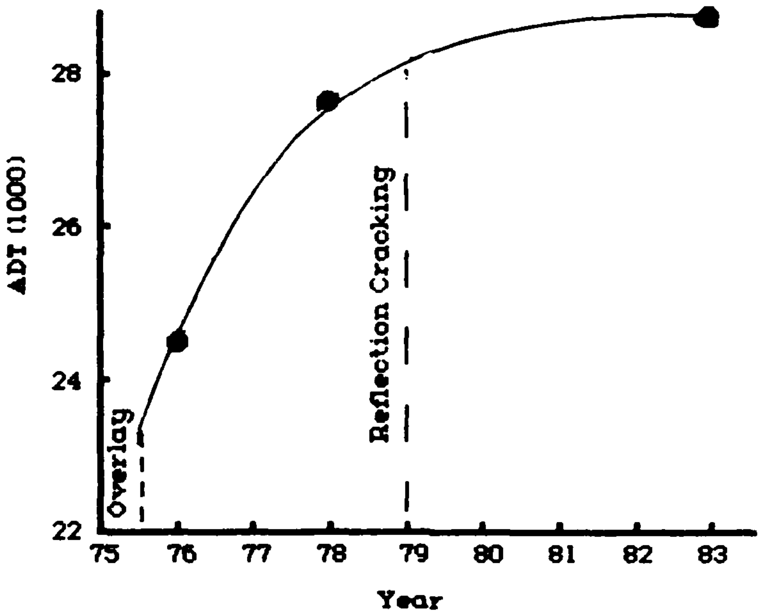


Figure A-4. ADT versus Time for SR 5 MP 85.3.

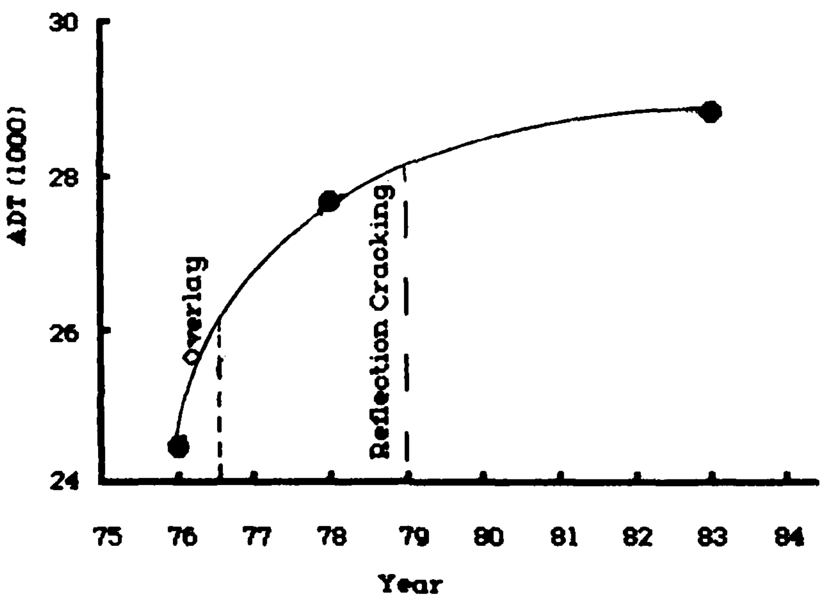


Figure A-5. ADT versus Time for SR 5 MP 86.7.

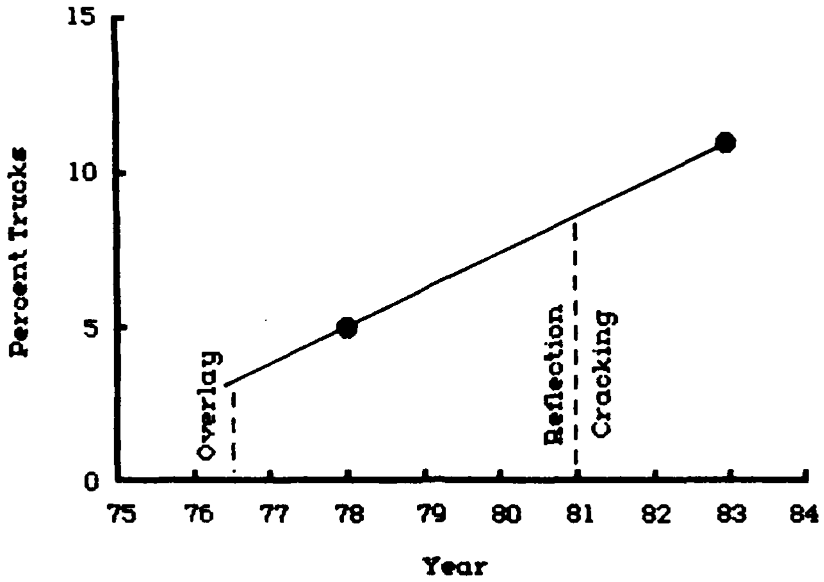
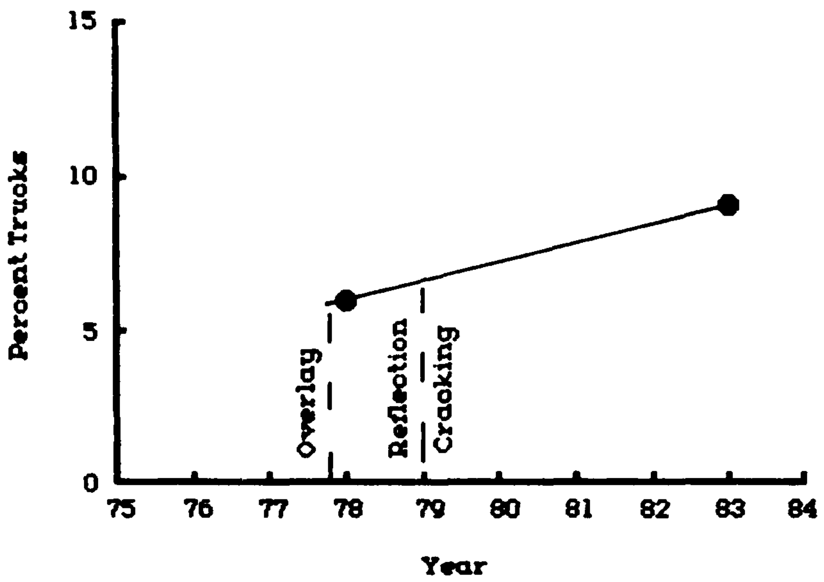


Figure A-6. Percent Trucks versus Time for SR 162.



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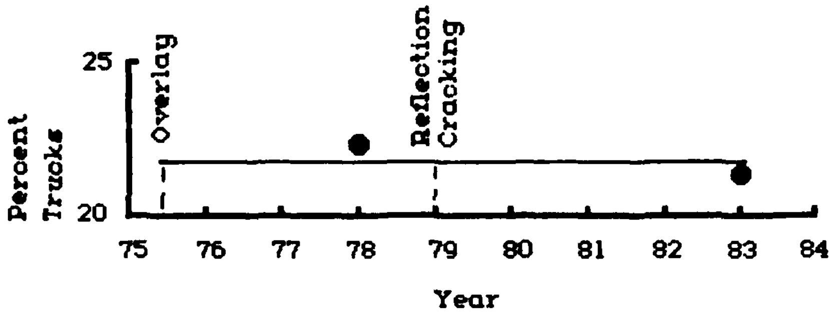

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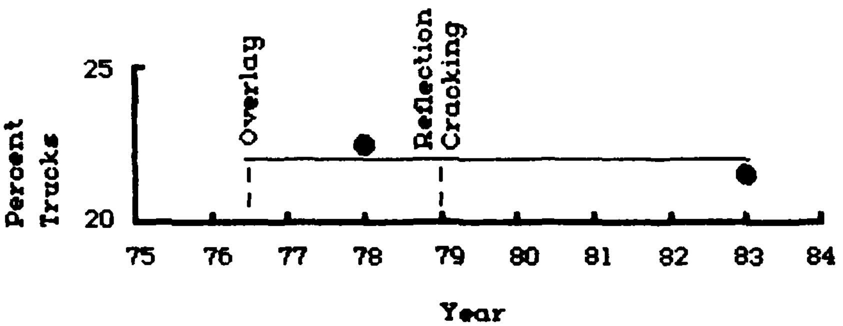

Figure A-9. Percent Trucks versus Time for SR 5 MP 86.7.

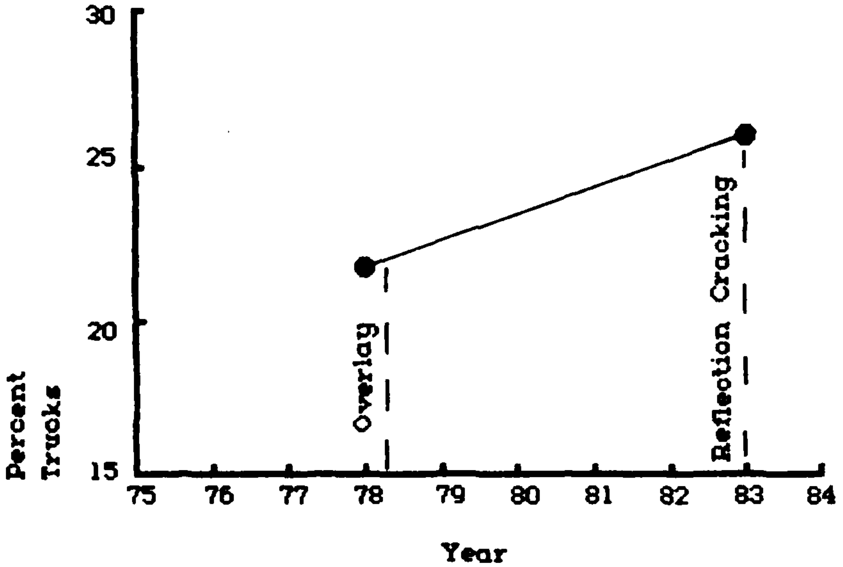

Figure A-10. Percent Trucks versus Time for SR 5 MP 73.1.

APPENDIX B

FWD DATA FOR TEST SECTIONS

TABLE B-1. FWD DATA FOR SR 162.

Slab No. and Locations	Applied Pressure, psi	Deflection at Distance, 10^{-3} in.			
		0.00 in.	11.81 in.	25.59 in.	47.24 in.
I Center	95.76	7.8	7.4	6.4	4.4
I Corner	93.44	15.0	-	-	-
II Center	94.74	6.7	6.1	5.4	3.8
II Corner	94.60	14.1	-	-	-
III Center	91.84	6.7	6.4	5.6	3.4
III Corner	93.14	15.2	-	-	-
IV Center	93.72	5.7	5.4	4.8	3.3
IV Corner	85.14	14.0	-	-	-
V Center	92.85	8.3	7.1	5.7	3.7
V Corner	80.63	18.8	-	-	-
VI Center	92.71	7.9	7.5	6.6	4.8
VI Corner	84.27	12.1	-	-	-

TABLE B-2. FWD DATA FOR SR 603.

Slab No. and Location	Applied Pressure, psi	Deflection at Distance, 10^{-3} in.			
		0.00 in.	11.81 in.	25.59 in.	47.24 in.
I Center	102.31	5.0	4.6	3.9	2.7
I Corner	78.74	11.6	-	-	-
II Center	100.13	6.5	5.3	4.0	2.4
II Corner	82.08	14.3	-	-	-
III Center	103.48	4.3	4.2	3.5	2.2
III Corner	85.87	8.8	-	-	-
IV Center	105.66	4.0	3.7	2.9	1.8
IV Corner	91.84	6.4	-	-	-
V Center	102.75	4.2	3.9	3.1	1.9
V Corner	91.84	5.9	-	-	-
VI Center	93.29	4.6	4.3	3.7	2.4
VI Corner	91.40	8.1	-	-	-

TABLE B-3. FWD DATA FOR SR 5 MP 73.1.

Slab No. and Location	Applied Pressure, psi	Deflection at Distance, 10^{-3} in.			
		0.00 in.	11.81 in.	25.59 in.	47.24 in.
I Center	106.24	2.6	2.2	1.9	1.4
I Corner	99.40	7.1	-	-	-
II Center	105.81	2.7	2.3	2.0	1.4
II Corner	102.31	8.3	-	-	-
III Center	107.99	2.5	2.2	2.0	1.4
III Corner	100.42	9.0	-	-	-
IV Center	109.85	2.5	2.2	1.9	1.3
IV Corner	105.37	6.6	-	-	-
V Center	107.55	2.4	2.2	1.9	1.3
V Corner	101.00	5.9	-	-	-
VI Center	108.57	2.4	2.2	2.0	1.4
VI Corner	100.57	5.7	-	-	-

TABLE B-4. FWD DATA FOR SR 5 MP 85.3.

Slab No. and Location	Applied Pressure, psi	Deflection at Distance, 10^{-3} in.			
		0.00 in.	11.81 in.	25.59 in.	47.24 in.
I Center	91.25	5.3	4.3	3.5	2.4
I Corner	94.74	7.7	-	-	-
II Center	99.40	3.1	2.8	2.5	2.0
II Corner	92.85	8.2	-	-	-
III Center	90.96	4.7	3.9	3.0	2.0
III Corner	95.32	8.7	-	-	-
IV Center	98.82	3.2	3.0	2.4	1.7
IV Corner	96.64	8.0	-	-	-
V Center	97.22	3.7	3.3	2.8	1.9
V Corner	92.85	10.0	-	-	-
VI Center	92.13	4.1	3.3	2.7	1.9
VI Corner	95.62	5.7	-	-	-

TABLE B-5. FWD DATA FOR SR 5 MP 86.7.

Slab No. and Location	Applied Pressure, psi	Deflection at Distance, 10^{-3} in.			
		0.00 in.	11.81 in.	25.59 in.	47.24 in.
I Center	110.90	2.5	2.1	1.9	1.3
I Corner	94.60	6.3	-	-	-
II Center	109.88	2.7	2.4	2.0	1.4
II Corner	97.22	5.7	-	-	-
III Center	106.24	2.6	2.3	2.0	1.3
III Corner	104.35	4.9	-	-	-
IV Center	111.19	2.4	2.2	1.9	1.3
IV Corner	101.58	8.9	-	-	-
V Center	108.28	2.7	2.4	2.0	1.4
V Corner	104.35	6.3	-	-	-
VI Center	106.68	2.4	2.2	1.9	1.3
VI Corner	97.51	7.0	-	-	-

APPENDIX C

LAYER MODULI CALCULATED BY BISDEF

TABLE C-1. LAYER MODULI CALCULATED BY BISDEF FOR SR 162.

Slab No.	E_1, psi	E_2, psi	E_3, psi
I	5.76×10^5	2.27×10^6	1.09×10^4
II	4.21×10^5	3.12×10^6	1.17×10^4
III	4.20×10^5	3.00×10^6	1.20×10^4
IV	5.87×10^5	3.28×10^6	1.37×10^4
V	3.18×10^5	5.34×10^6	1.28×10^4
VI	6.94×10^5	8.65×10^6	8.42×10^3

TABLE C-2. LAYER MODULI CALCULATED BY BISDEF FOR SR 603.

Slab No.	E_1, psi	E_2, psi	E_3, psi
I	1.32×10^6	9.66×10^6	1.80×1[illegible]
II	3.81×10^5	4.83×10^6	2.17×10^4
III	1.04×10^6	8.48×10^6	2.35×10^4
IV	6.83×10^5	9.37×10^6	3.09×10^4
V	8.30×10^5	7.55×10^6	2.80×10^4
VI	9.16×10^5	9.33×10^6	1.89×10^4

TABLE C-3. LAYER MODULI CALCULATED BY BISDEF FOR SR 5 MP 73.1.

Slab No.	E_1, psi	E_2, psi	E_3, psi
I	1.27×10^6	6.05×10^6	3.26×10^4
II	1.51×10^6	4.52×10^6	3.30×10^4
III	1.38×10^6	6.00×10^6	3.23×10^4
IV	1.32×10^6	4.14×10^6	3.97×10^4
V	1.23×10^6	4.52×10^6	3.70×10^4
VI	1.12×10^6	6.23×10^6	3.26×10^4
VII	1.89×10^6	6.66×10^6	3.05×10^4

TABLE C-4. LAYER MODULI CALCULATED BY BISDEF FOR SR 5 MP 85.3.

Slab No.	E_1, psi	E_2, psi	E_3, psi
I	5.73×10^5	2.43×10^6	1.81×10^4
II	2.00×10^6	1.07×10^7	1.80×10^4
III	1.00×10^6	1.47×10^6	2.29×10^4
IV	2.00×10^6	4.54×10^6	2.51×10^4
V	1.37×10^6	3.57×10^6	2.37×10^4
VI	6.20×10^5	3.85×10^6	2.33×10^4

TABLE C-5. LAYER MODULI CALCULATED BY BISDEF FOR SR 5 MP 86.7.

Slab No.	E_1, psi	E_2, psi	E_3, psi
I	1.59×10^6	5.19×10^6	3.78×10^4
II	1.67×10^6	3.68×10^6	3.55×10^4
III	1.47×10^6	3.34×10^6	3.70×10^4
IV	1.33×10^6	4.21×10^6	4.01×10^4
V	1.67×10^6	3.59×10^6	3.50×10^4
VI	1.55×10^6	4.27×10^6	3.65×10^4

TABLE C-6. EQUIVALENT PCC MODULUS AT CORNER OF EACH SLAB, PSI.

Test Section / Slab No.	SR 162	SR 603	SR 5 MP 73.1	SR 5 MP 85.3	SR 5 MP 86.7
I	510,000	56,000	180,000	610,000	170,000
II	610,000	50,000	120,000	255,000	260,000
III	470,000	115,000	92,000	215,000	410,000
IV	285,000	270,000	180,000	180,000	72,000
V	125,000	360,000	270,000	100,000	190,000
VI	610,000	270,000	380,000	1,150,000	100,000
VII	-	-	390,000	-	-

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APPENDIX D

CALCULATED STRAINS FOR OVERLAYS

TABLE D-1. OVERLAY STRAINS AND 18-KIP AXLE REPETITIONS FOR SR 162.

Slab No.	Bonded		Unbonded	
	ε, 10^{-6}in/in	No. 18-kip Axles	ε, 10^{-6}in/in	No. 18-kip Axles
I	*	-	104	29,698
II	*	-	108	35,558
III	*	-	108	32,949
IV	*	-	144	35,102
V	*	-	289	19,021
VI	*	-	88.9	33,250

*Strain values in compression.

TABLE D-2. OVERLAY STRAINS AND 18-KIP AXLE REPETITIONS FOR SR 603.

Slab No.	Bonded		Unbonded	
	ε, 10^{-6}in/in	No. 18-kip Axles	ε, 10^{-6}in/in	No. 18-kip Axles
I	149	34,687	205	39,091
II	237	33,601	415	16,064
III	106	76,586	193	14,062
IV	43.4	12,230	171	32,544
V	30.0	32,722	144	40,054
VI	47.	12,842	165	57,675

TABLE D-3. OVERLAY STRAINS AND 18-KIP AXLE REPETITIONS FOR SR 5 MP 73.1.

Slab No.	Bonded		Unbonded	
	ε, 10^{-6}in/in	No. 18-kip Axles	ε, 10^{-6}in/in	No. 18-kip Axles
I	56.2	3,842,196	105	3,444,891
II	67.7	3,645,781	104	3,744,753
III	80.2	3,798,685	115	3,550,542
IV	55.1	3,763,970	99.4	4,133.623
V	41.3	3,496,185	94.0	3,975,328
VI	29.7	4,224,556	90.2	3,365,522
VII	28.6	3,737,375	68.5	3,427,835

TABLE D-4. OVERLAY STRAINS AND 18-KIP AXLE REPETITIONS FOR SR 5 MP 85.3.

Slab No.	Bonded		Unbonded	
	ε, 10^{-6}in/in	No. 18-kip Axles	ε, 10^{-6}in/in	No. 18-kip Axles
I	*	-	114	3,152,109
II	40.3	2,579,270	81.8	4,135,812
III	51.9	2,975,716	121	3,230,760
IV	49.8	3,585,051	84.0	3,702,082
V	70.3	4,749,558	121	4,936,310
VI	*	-	84.7	2,704,768

*Strain values in compression.

TABLE D-5. OVERLAY STRAINS AND 18-KIP AXLE REPETITIONS FOR SR 5 MF 86.7.

Slab No.	Bonded		Unbonded	
	ε, 10^{-6}in/in	No. 18-kip Axles	ε, 10^{-6}in/in	No. 18-kip Axles
I	53.2	2,059,999	89.8	3,765,773
II	39.6	3,805,377	79.0	2,203,708
III	27.1	3,782,104	73.2	4,177,776
IV	88.8	2,991,931	119	4,084,472
V	49.1	1,417,727	85.9	1,584,956
VI	71.6	1,889,040	103	1,587,515

APPENDIX E

REFLECTION CRACKING SURVEY

FOR

AIR FORCE PAVEMENT ENGINEERS
(Previously sent to all Air Force Installations)

AIRFIELD PAVEMENT REFLECTION CRACKING QUESTIONNAIRE

DEFINITION OF THE PROBLEM

Pavement reflection cracking is cracking of a resurface or overlay above underlying cracks or joints. More specifically, reflection cracking is often described as fractures (or cracks) in an asphalt concrete overlay which are due to movements of the underlying portland cement concrete which can be due to traffic loads and/or environmental causes (such as thermal expansion/contraction, warping of slab edges, etc.).

1. **RESPONDENT INFORMATION**

Name____________________________ Date____________________

Title____________________________ Phone (   )______________

Address____________________________________

____

2. **GENERAL**

(a) *Does your base have any asphalt* concrete overlays on portland cement concrete pavements? Yes No

(b) If the answer to 2(a) is yes, please provide the following information (multiple facilities are appropriate):

Type of Pavement Facility(1)	Traffic(2)	Age(3)	Asphalt Concrete Overlay Thickness (in.)	Original PCC Thickness in.	Overall Condition(4)	Sererity of Reflection Cracking(5)

Notes: (1) Type of Facility: runway, taxiway, apron, etc.

(2) Traffic: this category relates to how much traffic uses the facility (heavy, medium, light, none).

(3) Age: time since PCC was overlayed to date.

(4) Overall Condition: relates to overall surface condition of the facility (good, fair, poor). Please note the predominant type of surface distress (such as alligator cracking, transverse cracking, longitudinal cracking, rutting, etc.). Please use the PCI if one exists.

(5) Severity of Reflection Cracking: relates to whether reflection cracking exists for the facility listed. For severity of reflection cracking, you may use terms such as low, medium, and high as defined by AFR 93-5, Chapter 3. Please provide an approximate percentage if the facility area is affected by reflection cracking.

(c) Any additional comments you would like to make about these facilities.

3. DESIGN PRACTICE

(a) How does your installation design asphalt concrete overlays for old portland concrete pavements:

(i) AFM 88-6

(ii) COE Design Guide

(iii) Asphalt Institute Design

(iv) State or Local Government Manuals

(v) Other, please describe

(b) What do you think should be the primary concern in designing asphalt concrete overlays for portland cement concrete pavements?

(i) Reduce reflection cracking resulting in possible FOD damage? Yes No

(ii) Reduce reflection cracking resulting in increased pavement maintenance? Yes No

(iii) Provide adequate overall pavement load capacity to accommodate the mission aircraft? Yes No

(iv) Other concerns, please describe:

(c) Are you satisfied with the available asphalt concrete overlay design methods? Yes No

(i) If not satisfied with available design methods, what changes to such methods would you suggest?

4. **METHODS FOR CONTROLLING REFLECTION CRACKING**

(a) Some of the methods which have been used to control reflection cracking can be categorized as follows:

(i) Treatments of existing pavement

*Breaking and seating PCC slabs

*Subsealing

(ii) Use of interlayers

*Stress-absorbing membranes such as asphalt- rubber

*Fabrics (such as Petromat, etc.)

*Bond breakers (such as sand, fabrics, etc.)

(iii) Cushion courses

(iv) Application of thicker asphalt concrete overlays

(v) Other

(b) Has your installation used any of the above methods listed in 4(a) at your base and, if so, please describe your satisfaction with the methods.

(c) Does your installation plan to use any of the methods listed in 4 (a)? If so, when do you estimate the work will be accomplished?

Your assistance in providing this information is appreciated.